CHAPTER 7

HYDROLOGY

CHAPTER 7 – TABLE OF CONTENTS

7.1 HYDROLOGIC DESIGN POLICIES

7.1.1 Introduction

Following is a summary of policies that shall be followed for hydrologic analysis. For a more detailed discussion, refer to Chapter 2 of Reference *(1)*.

7.1.2 Site Data

Because hydrologic considerations can influence the selection of a highway corridor and the alternative routes within the corridor, studies and investigations, including consideration of the environmental and ecological impact of the project, shall be undertaken. Also, special studies and investigations may be required at sensitive locations. The magnitude and complexity of these studies shall be commensurate with the importance and magnitude of the project and the problems encountered. Typical data to be included in such surveys or studies are topographic maps, aerial photographs, streamflow records, historical high-water elevations, flood discharges and locations of hydraulic features (e.g., reservoirs, water projects, designated or regulatory floodplain areas).

7.1.3 Flood Hazards

A hydrologic analysis is a prerequisite to identifying special flood hazard areas. Then, hydraulic structures may be designed that are cost effective, require a minimum amount of maintenance and provide safety to the traveling public.

7.1.4 Coordination

Interagency coordination is desirable and often necessary because many levels of government plan, design and construct highway and water resource facilities that might have an effect on each other. In addition, agencies can share data and experiences within project areas to assist in the completion of accurate hydrologic analysis.

7.1.5 Documentation

Experience indicates that the design of highway drainage facilities should be adequately documented. Frequently, it is necessary to refer to plans and specifications long after the actual construction has been completed. Thus, it is necessary to fully document the results of all hydrologic analysis.

7.1.6 Evaluation Of Runoff Factors

For all hydrologic analyses, the following factors shall be evaluated and included when they will have a significant effect on the final results:

- drainage basin characteristics including size, shape, slope, land use, geology, soil type, surface infiltration and storage;
- stream channel characteristics including geometry and configuration, natural and artificial controls, channel modification, aggradation/degradation and ice and debris;
- floodplain characteristics; and

- meteorological characteristics such as precipitation amounts and type (rain, snow, hail or combinations thereof), storm cell size and distribution characteristics, storm direction and time rate of precipitation (hyetograph).

7.1.7 Flood History

All hydrologic analysis shall consider the flood history of the area and the effect of these historical floods on existing and proposed structures. The flood history shall include the historical floods and the flood history of any existing structures.

7.1.8 Hydrologic Methods

Many hydrologic methods are available. The methods to be used and the circumstances for their use are listed below. If possible, the method should be calibrated to local conditions and tested for accuracy and reliability.

Following is a list of approved methods:

- Generally, the Rational method should not be used for drainage areas in excess of 300 acres.

- Estimates of 10-, 25-, 50- and 100-yr peak discharge values for ungaged, unregulated watersheds in the Virgin River basin can be developed using the regression equations described at : http://www.udot.utah.gov/index.php/m=c/tid=285. As additional state regression equations become available they will be published in this manual.

- Log-Pearson III analyses shall be used for all routine designs, provided that there is at least 10 yrs of a continuous or synthesized record for 10-yr discharge estimates and 25 yrs for 100-yr discharge estimates.

- State hydrographs shall be used for all routine rural designs.

- NRCS and other unit hydrograph methods.

- Suitable computer programs (e.g., WMS, HYDRO, HEC-1, HEC-HMS and TR-20) may be used to facilitate tedious hydrologic calculations.

- The 100-yr discharges specified in the FEMA flood insurance study shall be used to analyze the impacts of a proposed crossing on a regulatory floodway. However, if these discharges are deemed to be outdated, the discharges based on current methods may be used subject to receipt of necessary regulatory approvals.

7.1.9 Design Frequency

A design frequency should be selected commensurate with the facility’s cost, traffic volume, potential flood hazard to property, expected level of service, political considerations and budgetary constraints plus the magnitude and risk associated with damages from larger flood events. For long highway routes with no practical detour, where many sites are subject to independent flood events, it may be necessary to increase the design frequency at each site to avoid frequent route interruptions from floods. In selecting a design frequency, potential upstream land use should be considered which could reasonably occur over the anticipated life of the drainage facility.

7.1.10 Economics

Hydrologic analysis should include the determination of several design flood frequencies for use in the hydraulic design. These frequencies are used to size different drainage facilities to allow for an optimum design, which considers both risk of damage and construction cost. Consideration should be given to the flood frequency used to design other structures along a highway corridor.

7.1.11 Review Frequency

All proposed structures obtained using the selected design frequency shall be reviewed using a base flood and/or superflood to ensure that there are no unexpected flood hazards.

7.2 OVERVIEW

7.2.1 Introduction

The analysis of the peak rate of runoff, volume of runoff and time distribution of flow is fundamental to the design of drainage facilities. Errors in the estimates will result in a structure that is either undersized and causes more drainage problems or oversized and costs more than necessary. In contrast, it must be realized that any hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex, and too little data are available on the factors influencing the rural and urban rainfall-runoff relationship to expect exact solutions.

7.2.2 Definition

Hydrology is generally defined as a science that addresses the interrelationship between water on and under the earth and in the atmosphere. For this *Manual*, hydrology will address estimating flood magnitudes as the result of precipitation. In the design of highway drainage structures, floods are usually considered in terms of peak runoff or discharge in cubic feet per second (ft^3/s) and hydrographs as discharge per time. For structures that are designed to control the volume of runoff (e.g., detention storage facilities) or where flood routing through culverts is used, then the entire discharge hydrograph will be of interest. Wetland Hydrology, the water-related driving force to create wetlands, is addressed in Chapter 10 of Reference *(1)*.

7.2.3 Factors Affecting Floods

In the hydrologic analysis for a drainage structure, it must be recognized that there are many variable factors that affect floods. These are some of the factors to be recognized and considered, individually, site by site:

- rainfall amount and storm distribution;
- drainage area size, shape and orientation;
- ground cover and soil type;

- slopes of terrain and stream(s);
- antecedent moisture condition;
- storage potential (overbank, ponds, wetlands, reservoirs and channel);
- watershed development potential;
- type of precipitation (rain, snowmelt, hail or combinations thereof); and
- elevation and mixed-population events.

7.2.4 Sources of Information

The type and source of information available for hydrologic analysis will vary from site to site, and it is the responsibility of the designer to determine what information is available and applicable to a particular analysis. A comprehensive list of data sources is included in the Data Collection Chapter.

7.3 SYMBOLS AND DEFINITIONS

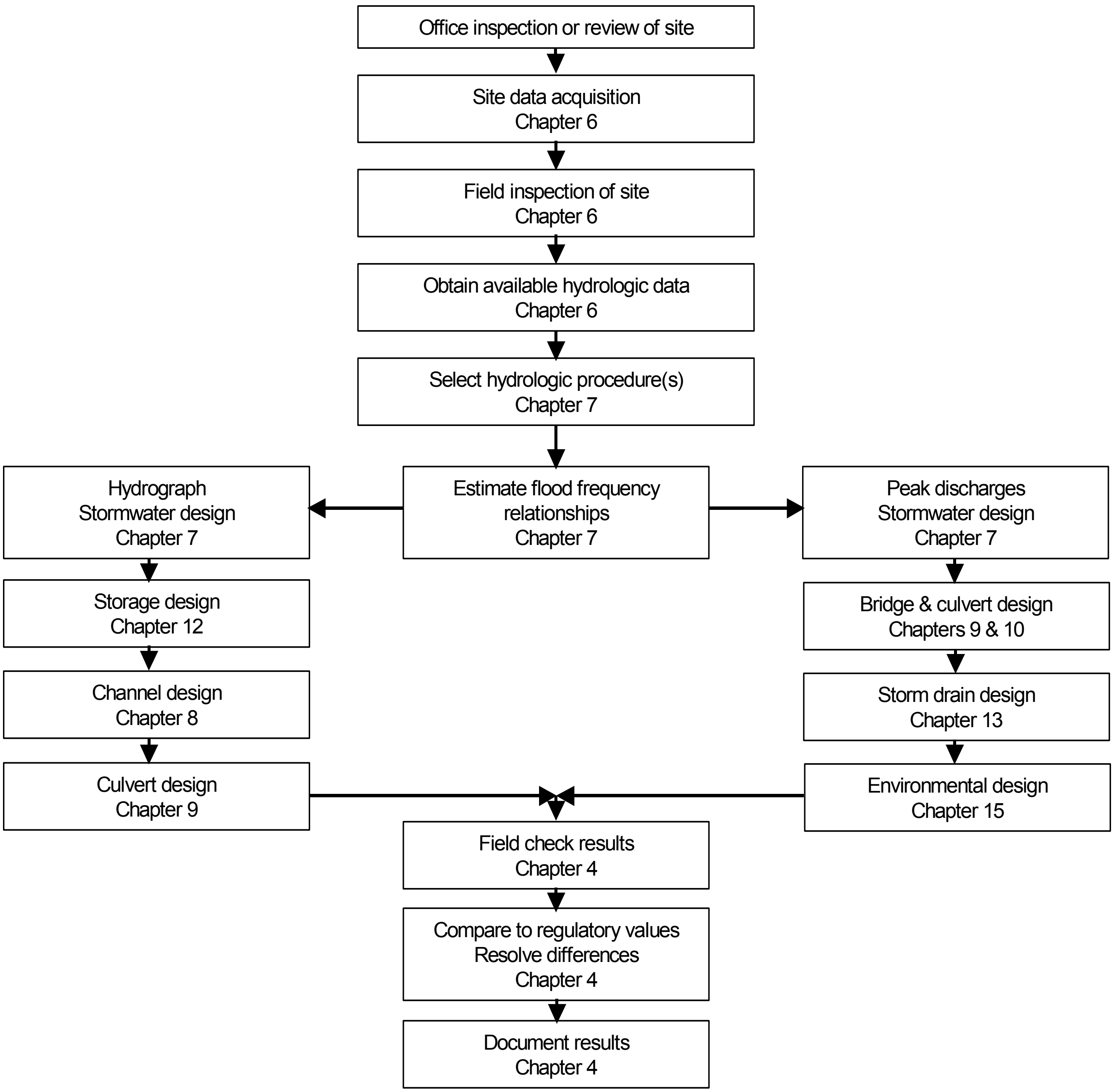
To provide consistency within this chapter and throughout this *Manual,* the symbols in Table 7-1 will be used. These symbols have been selected because of their wide use in hydrologic publications.

TABLE 7-1 — Symbols and Definitions

Symbol	Definition	Units
A	Drainage area	acre, mi^2
BDF	Basin development factor	%
C	Runoff coefficient	-
C_f	Frequency factor	-
CN	NRCS-runoff curve number	-
d	Time interval	h
DH	Difference in elevation	ft
I	Rainfall intensity	in/h
IA	Percentage of impervious area	%
I_a	Initial abstraction from total rainfall	in
K	Frequency factor for a particular return period and skew	-
L	Lag	h
l	Length of mainstream to furthest divide	ft
L_{ca}	Length along main channel to a point opposite the watershed centroid	mi
M	Rank of a flood within a long period	-
n	Manning roughness coefficient	-
N	Number of years of flood record	yrs
P	Accumulated rainfall	in
Q	Rate of runoff	ft^3/s
q	Storm runoff during a time interval	in
R	Hydraulic radius	ft
RC	Regression constant	-
RQ	Equivalent rural peak runoff rate	ft^3/s
S or Y	Ground slope	ft/ft, ft/mi or %
S	Potential maximum retention storage	in
NRCS	Natural Resources Conservation Service	-
SL	Main channel slope	ft/ft
S_L	Standard deviation of the logarithms of the peak annual floods	-
ST	Basin storage factor	%
T_B	Time base of unit hydrograph	h
t_c or T_c	Time of concentration	m or h
T_L	Lag time	h
UQ	Urban peak runoff rate	ft^3/s
V	Velocity	ft/s
X	Logarithm of the annual peak	-

7.4 HYDROLOGIC ANALYSIS PROCEDURE FLOWCHART

The hydrologic analysis procedure flowchart shows the steps needed for the hydrologic analysis and the designs that will use the hydrologic estimates.



7.5 CONCEPT DEFINITIONS

Following are discussions of concepts that will be important in a hydrologic analysis. These concepts will be used throughout this Chapter to address different aspects of hydrologic studies:

Antecedent Moisture Conditions — Antecedent moisture conditions are the soil moisture conditions of the watershed at the beginning of a storm. These conditions affect the volume of runoff generated by a particular storm event. Notably, they affect the peak discharge only in the lower range of flood magnitudes, say, below approximately the 15-yr event threshold. As floods become more rare, antecedent moisture has a rapidly decreasing influence on runoff.

Depression Storage — Depression storage is the natural depressions within a watershed that store runoff. Generally, after the depression storage is filled, runoff will commence.

Frequency — Frequency is the number of times a flood of a given magnitude can be expected to occur on average over a long period of time. Frequency analysis is then the estimation of peak discharges for various recurrence intervals. Another way to express frequency is with probability. Probability analysis seeks to define the flood flow with a probability of being equaled or exceeded in any year.

Hydraulic Roughness — Hydraulic roughness is a composite of the physical characteristics that influence the flow of water across the earth's surface, whether natural or channelized. It affects both the time response of a watershed and drainage channel and the channel storage characteristics.

Hydrograph — The hydrograph is a graph of the time distribution of runoff from a watershed.

Hyetographs — The hyetograph is a graph of the time distribution of rainfall over a watershed.

Infiltration — Infiltration is a complex process of allowing runoff to penetrate the ground surface and flow through the upper soil surface. The infiltration curve is a graph of the time distribution at which this occurs.

Interception — Storage of rainfall on foliage and other intercepting surfaces during a rainfall event is called interception storage.

Lag Time — The lag time is defined as the time from the centroid of the excess rainfall to the peak of the hydrograph.

Peak Discharge — The peak discharge, sometimes called peak flow, is the maximum rate of flow of water passing a given point during or after a rainfall event or snowmelt.

Rainfall Excess — The rainfall excess is the water available to runoff after interception, depression storage and infiltration have been satisfied.

Stage — The stage of a river is the elevation of the water surface above some elevation datum.

Time of Concentration — The time of concentration is the time it takes the drop of water falling on the hydraulically most remote point in the watershed to travel through the watershed to the outlet.

Unit Hydrograph — A unit hydrograph is the direct runoff hydrograph resulting from a rainfall event that has a specific temporal and spatial distribution, lasts for a specific duration and has unit volume (or results from a unit depth of rainfall). The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to one millimeter of runoff from the drainage area. When a unit hydrograph is shown with units of cubic meters per second, it is implied that the ordinates are cubic meters per second per millimeter of direct runoff.

References — For a more complete discussion of these concepts and others related to hydrologic analysis, the reader is referred to Chapter 2 of Reference *(1)* and Reference *(3)*.

7.6 DESIGN FREQUENCY

7.6.1 Overview

Because it is not economically feasible to design a structure for the maximum runoff a watershed is capable of producing, a design frequency must be established. The frequency with which a given flood can be expected to occur is the reciprocal of the probability or chance that the flood will be equaled or exceeded in a given year. If a flood has a 20% chance of being equaled or exceeded each year over a long period of time, the flood will be equaled or exceeded on an average of once every five years. This is called the return period or recurrence interval (RI). Thus, the exceedence probability equals 100/RI.

The designer should note that the 5-yr flood is not one that will necessarily be equaled or exceeded every five years. There is a 20% chance that the flood will be equaled or exceeded in any year; therefore, the 5-yr flood could conceivably occur in several consecutive years. The same reasoning applies to floods with other return periods.

7.6.2 Design Frequency

7.6.2.1 Cross Drainage

A drainage facility shall be designed to accommodate a discharge with a given return period(s) for the following circumstances. The design shall be such that the backwater (the headwater) caused by the structure for the design storm does not:

- increase the flood hazard significantly for property,
- overtop the highway, or
- exceed a certain depth on the highway embankment.

Based on these design criteria, a design involving temporary roadway overtopping for floods larger than the design event is acceptable practice. Usually, if overtopping is allowed, the structure may be designed to accommodate a flood of some lower frequency without overtopping.

7.6.2.2 Storm Drains

A storm drain shall be designed to accommodate a discharge with a given return period(s) for the following circumstances. The design shall be such that the storm runoff does not:

- increase the flood hazard significantly for property;
- encroach onto the street or highway to cause a significant traffic hazard; or
- limit traffic, emerging vehicle or pedestrian movements to an unreasonable extent.

Based on these design criteria, a design involving temporary street or road inundation for floods larger than the design event is acceptable practice.

7.6.3 Review Frequency

After sizing a drainage facility using a flood and sometimes the hydrograph corresponding to the design frequency, it shall be necessary to review this proposed facility with a base discharge. This is done to ensure that there are no unexpected flood hazards inherent in the proposed facility(ies). The review flood shall be the 100- yr event. In some cases, a flood event larger than the 100-yr flood is used for analysis to ensure the safety of the drainage structure and downstream development.

7.6.4 Frequency Table

Appendix 7.A presents a guide of preferred design frequencies to be used by the Department for the various drainage facilities on streets and highways.

7.6.5 Rainfall vs. Flood Frequency

Drainage structures are designed based on some flood frequency. However, certain hydrologic procedures use rainfall and rainfall frequency as the basic input. Thus, it is commonly assumed that the 10-yr rainfall will produce the 10-yr flood. Depending on antecedent soil moisture conditions and other hydrologic parameters, this may be true or there may not be a direct relationship between rainfall and flood frequency.

7.6.6 Rainfall Curves

Rainfall data are available for many geographic areas. From these data, rainfall-intensity-duration curves have been developed for the commonly used design frequencies. Appendix 7.B contains the curves available at this time for the Department.

7.6.7 Discharge Determination

Estimating peak discharges of various recurrence intervals is one of the most common engineering challenges faced by drainage facility designers. The problem can be divided into two general categories:

- *Gaged Sites* — The site is at or near a gaging station and the streamflow record is of sufficient length to be used to provide estimates of peak discharges. A complete record is defined as one having at least 25 yrs of continuous or synthesized data. In Utah, this may be a relatively rare situation.

- *Ungaged Sites* — The site is not near a gaging station and no streamflow record is available. This situation is very common in Utah.

This Chapter will address hydrologic procedures that can be used for both categories.

7.7 HYDROLOGIC PROCEDURE SELECTION

7.7.1 Overview

Streamflow measurements for determining a flood frequency relationship at a site are usually unavailable; in such cases, it is accepted practice to estimate peak runoff rates and hydrographs using statistical or empirical methods. In general, results from using several methods should be compared, not averaged. The Department practice shall be to use the discharge that best reflects local project conditions with the reasons documented. The Department use for each procedure is outlined with each hydrologic procedure given below.

7.7.2 Peak Flow Rates or Hydrographs

A consideration of peak runoff rates for design conditions is generally adequate for conveyance systems (e.g., storm drains or open channels). However, if the design must include flood routing (e.g., storage basins or complex conveyance networks), a flood hydrograph is usually required. Although the development of runoff hydrographs (typically more complex than estimating peak runoff rates) is often accomplished using computer programs, some methods are adaptable to nomographs or other desktop procedures.

7.7.3 Hydrologic Procedures

The methods presented in this Chapter were selected to be consistent with the methods available in the computer program HYDRAIN Integrated Drainage Design Computer System, HEC 1 and HMS. The hydrologic model within the HYDRAIN system is called HYDRO, and this model allows the user to select among several hydrologic procedures:

ANALYSIS OF STREAM GAGE DATA. Where stream gage data are available, they can be used to develop peak discharges and hydrographs.

LOG-PEARSON III FLOOD FREQUENCY. With at least 25 yrs of continuous or synthesized stream gage data, the Log-Pearson III is considered to be a reliable method for estimating flood frequency relationships where no significant changes in the watershed have occurred or are anticipated. Data can be obtained from the local USGS office in your area.

REGRESSION EQUATIONS. Peak flow can be calculated by using regression equations developed for specific geographic regions. The equations are in the form of a log-log formula, where the dependent variable would be the peak flow for a given frequency, and the independent variables may be variables (e.g., area, slope, channel geometry, other meteorological, physical or site specific data).

RATIONAL METHOD. Provides peak runoff rates for small urban and rural watersheds less than 300 acres, but it is best suited to urban storm drain systems. It should be used with caution if the time of concentration exceeds 30 min. Rainfall is a necessary input.

NRCS SYNTHETIC UNIT HYDROGRAPH. The Natural Resources Conservation Service has developed a synthetic unit hydrograph procedure that has been widely used for developing rural and urban hydrographs. The unit hydrograph used by the NRCS method is based upon an analysis of a large number of natural unit hydrographs from a broad cross section of geographic locations and hydrologic regions. Rainfall is a necessary input.

UNIT HYDROGRAPH ANALYSIS. This is a technique to approximate the rainfall-runoff response of typical watersheds. The key to analyzing unit hydrographs is selecting the correct rainfall events. The selected storms must be representative of the temporal and spatial distribution of rainfall that is characteristic of storms resulting in peak discharges of the magnitudes and frequency selected for design.

7.8 CALIBRATION

7.8.1 Definition

Calibration is a process of varying the parameters or coefficients of a hydrologic method so that it will estimate peak discharges and hydrographs consistent with recorded local rainfall and streamflow data.

Figure 7-1 is an illustration of a hydrograph resulting from flow data as compared to a hydrograph resulting from using a non-calibrated and calibrated hydrologic procedure. It can be seen that the calibrated hydrograph, although not exactly duplicating the hydrograph from streamflow data, is a much better representation of the streamflow hydrograph than the non-calibrated hydrograph.

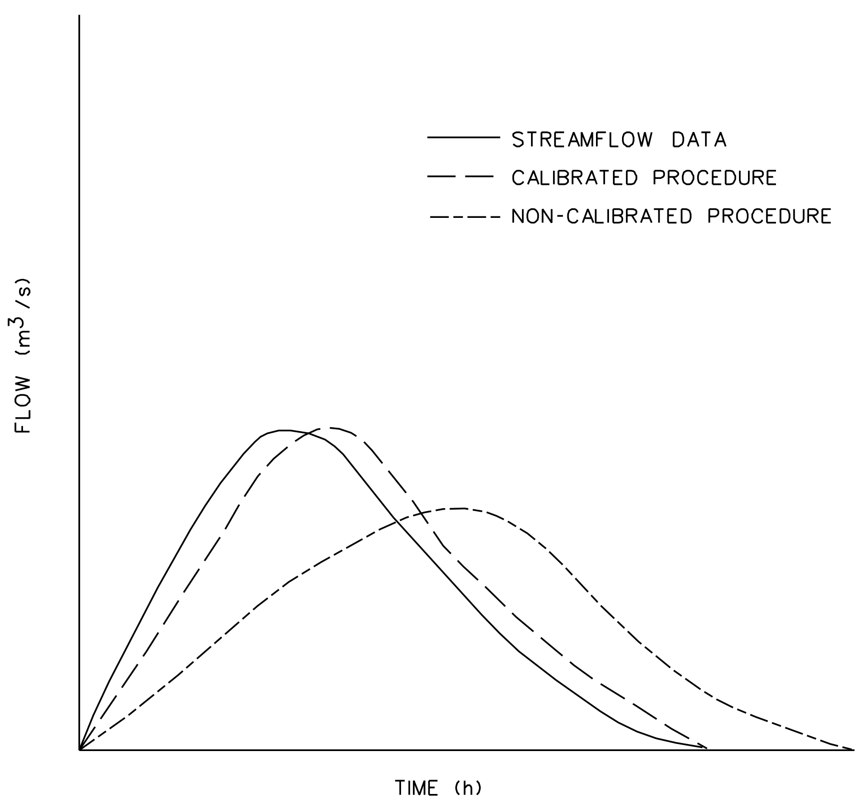


FIGURE 7-1— Calibrated Hydrograph

7.8.2

Hydrologic Accuracy

The accuracy of the hydrologic estimates will have a major effect on the design of drainage or flood control facilities. Although it might be argued that one hydrologic procedure is more accurate than another, practice has shown that all of the methods discussed in this Chapter can, if calibrated, produce acceptable results consistent with observed or measured events. What should be emphasized is the need to calibrate the method for local conditions. This calibration process can result in much more accurate and consistent estimates of peak flows and hydrographs.

7.8.3 Calibration Process

The calibration process can vary depending on the data or information available for a local area:

1. If streamflow data are available for an area, the hydrologic procedures can be calibrated to these data. The process would involve generating peak discharges and hydrographs for different input conditions (e.g., slope, area, antecedent soil moisture conditions) and comparing these results to the gaged data. Changes in the model would then be made to improve the estimated values as compared to the measured values.

2. After changing the variables or parameters in the hydrologic procedure, the results should be checked against another similar gaged stream or another portion of the streamflow data that were not used for calibration.

3. If some local agency has developed procedures or equations for an area based on streamflow data, general hydrologic procedures can be calibrated to these local procedures. In this way, the general hydrologic procedures can be used for a greater range of conditions (e.g., land uses, size, slope).

4. The calibration process should only be undertaken by personnel highly qualified in hydrologic procedures and design.

5. If reasonable values do not produce reasonable results, then the model should be questioned and its use carefully considered (e.g., having to use terrain variables that are obviously dissimilar to the geographic area to calibrate to measured discharges or hydrographs).

7.9 ANALYSIS OF STREAM GAGE DATA

7.9.1 Introduction

Many gaging stations exist throughout Utah where data can be obtained and used for hydrologic studies. If a project is located near one of these gages and the gaging record is of sufficient length in time, a frequency analysis may be made according to the following discussion. The most important aspect of applicable station records is the series of annual peak discharges. It is possible to apply a frequency analysis to that data for the derivation of flood-frequency curves. Such curves can then be used in several different ways:

- If the subject site is at or very near the gaging site and on the same stream and watershed, the discharge for a specific frequency from the flood-frequency curve can be used directly.

- If the facility site is nearby or representative of a watershed with similar hydrologic characteristics, transposition of frequency discharges is possible.

- If the flood-frequency curve is from one of a group of several gaging stations comprising a hydrologic region, then regional regression relations may be derived. Regional regression relations are usually furnished by established hydrologic agencies, and the designer will not be involved in their development.

Two methods of estimating flood frequency curves from stream gage data are provided in this *Manual*. The first method is a graphical procedure based on the Gumbel method. The second, a statistical method, makes use of the Log-Pearson Type III frequency distribution.

7.9.2 Application

The Department shall use the stream gage analysis findings for design when there are sufficient years of measured or synthesized stream gage record and no significant changes have occurred or are anticipated in the watershed. The preferred method for analyzing stream gage data is the Log-Pearson Type III distribution method with the Gumbel graphical method being used primarily as a check to help identify potential errors, particularly with the frequency estimates of larger floods. Where significant discrepancies 20% + are encountered in the findings between the two methods, special studies shall be required. These special studies shall consist of such functions as comparison with regression equations, particularly channel geometry characteristics equations, application of other flood-frequency methods, and the collection and analysis of historical data. Outliers shall be placed into perspective using the procedure found in Bulletin #17B *(5)*.

7.9.3 Graphical Method Procedure

The gaging station or stations most representative of a site that are in the vicinity or hydrologic region of a proposed structure should be selected. A record of 25 years or more is considered desirable. Stations with records that include controlled watershed runoff should be avoided because the natural flood events may not have been recorded. The following Steps may then be taken to develop a flood-frequency curve representing the data:

Step 1 The peak discharges for each water year are listed in chronological order. A water year extends from October 1 through September 30.

Step 2 After the discharges have been listed, they are then numbered in order of their magnitude; that is, the highest discharge for a particular gaging station is assigned the number 1, the next highest 2 and so on. The numbering system will indicate the relative distribution of floods during a given period of years.

Step 3 The probability of exceedence, also called the plotting position, for each annual peak flow is computed using the formula:

$$\text{Plotting Position} = M/(N + 1) \tag{7.1}$$

where: N = number of years of record
M = rank of a given flood beginning with 1

The reciprocal of the plotting position, (N + 1)/M, is the recurrence interval in years.

The recurrence interval vs. discharge for each year is plotted on commercially available log-probability paper, with the recurrence interval or plotting position as the x-axis and the magnitude of the associated discharge as the y-axis log scale. The data plots shown on the graph illustrate the frequency distribution of the floods for a given station.

Step 5 If Gumbel paper is used, the points should theoretically tend to fall in a straight line when the flow records are normally distributed. This special paper has been developed so that sample data will plot as a straight line if the data are distributed according to the formula:

$$F(Q) = e^{-x} \tag{7.2}$$

where:

$x = e^{\alpha(Q - \beta)}$
$\alpha = 1.281/S$
$\beta = Q' - 0.450\ S$
Q' = mean flow
Q = mean peak
S = standard deviation

This paper is not available commercially, but most USGS offices have prepared forms of the paper on which the horizontal scale has been transformed by the double-logarithmic transform of Equation 7.2.

Step 6 The recurrence interval and corresponding discharge is considered reasonable within the range of plotted data points. Extrapolation beyond the limits of the plotted points is not recommended.

Step 7 This frequency curve applies only to the point on the stream at which the gage is located. The use of the flood frequency curve is as follows: enter with the desired frequency or RI and read up to the geometric mean line, then move across to the discharge scale for the design discharge. The reverse procedure is used when a discharge is known, and information on frequency is desired.

Step 8 Flood frequency curves can also be plotted with stream stage rather than discharge as the ordinate. This is somewhat more convenient when checking high-water elevations at a bridge site.

The analysis of gaged data permits an estimate of the peak discharge for the desired return period at a particular site. A best-fit line can be drawn through the data points by eye, and the peak flow corresponding to the desired return period could be extracted from the curve. This is a very subjective method, and each designer may derive different estimates from the same data set. Experience has shown that statistical frequency distributions may be more representative of naturally occurring floods and can be reliable when used for prediction. Although several different distributions are used for frequency analysis, experience has shown the Log-Pearson Type III distribution to be one of the most useful. The Log-Pearson III distribution and the process of fitting it to a particular data sample are described in detail in Bulletin #17B (Reference *(5)*). The following abbreviated procedure is taken from that publication.

In the course of preparing a frequency analysis for a particular watershed, the designer will undoubtedly encounter situations where further adjustments to the data are necessary. Special handling of outliers, historical data, incomplete data and zero flow years is covered in detail in Bulletin #17B.

The computer system HYDRAIN provides the Log-Pearson III flood frequency analysis. The analysis follows the Bulletin #17B guidelines for the calculation of a Log-Pearson frequency curve based on the mean, standard deviation and skewness of the logarithms of the recorded annual peak flows.

7.9.4 Statistical Method Procedure

The Log-Pearson Type III distribution is the recommended statistical method. This method is defined by three standard statistical parameters — the mean, standard deviation and coefficient of skew. These parameters are determined from the data sample, which normally consists of the peak annual flows for a period of record. Formulas for the computation of these parameters are given below:

$$Q = (\Sigma X) / N \quad \text{(mean of logs)} \tag{7.3}$$

where: N = number of observations and X is the logarithm of the annual peak.

The standard deviation of logs is:

$$SL = \{[\Sigma X^2 - (\Sigma X)^2 / N] / [N - 1]\}^{1/2} \tag{7.4}$$

The coefficient of skew of logs is:

$$G = [N^2(\Sigma X^3) - 3N(\Sigma X)(\Sigma X^2) + 2(\Sigma X)^3] / [N(N - 1)(N - 2) S_L^3] \tag{7.5}$$

Using these three parameters, the magnitude of the flood of the desired frequency can be determined from the Equation:

$$\log Q = Q_L + KS_L \tag{7.6}$$

where:

- log Q = the logarithm of the flood magnitude
- Q_L = the mean of the logarithms of the peak annual floods
- K = a frequency factor for a particular return period and coefficient of skew (values of K for different coefficients of skew and return periods are given in Bulletin #17B)
- S_L = the standard deviation of the logarithms of the peak annual flood

If a flood frequency curve is necessary, then by computing several values of Q for different return periods, the Log-Pearson fit to the data can be plotted on standard log probability paper. If the skew of the sample data happens to be equal to zero, the plot of the Log-Pearson fit to the data will be a straight line. If the skew is negative, the plot will be a curve with a downward concavity. If the skew is positive, the plot will be a curve with upward concavity.

7.9.5 Skew

There are three alternative methods for determining the value of the skew coefficient to be used in calculating the Log-Pearson curve fit as follows:

1. Station Skew. The value of skew that is calculated directly from the gage data using the above formula. This value may not be a true representation of the actual skew of the data if the period of record is short or if there are extreme events in the period of record.

2. Generalized Skew. Bulletin #17B *(5)* contains a map of generalized skew coefficients of the logarithms of annual maximum streamflows throughout the United States and average skew coefficients by one degree quadrangles over most of the country.

3. Weighted Skew. Often the station skew and the generalized skew can be combined to provide a better estimate for a given sample of flood data. Bulletin #17B outlines a procedure for combining the station skew and the generalized skew to provide a weighted skew.

7.9.6 Transposition of Data

The transposition of design discharges from one basin to another basin with similar hydrologic characteristics is accomplished by multiplying the design discharge by the direct ratio of the respective drainage areas raised to the power shown in Table 7-2. Thus, on streams where no gaging station is in existence, records of gaging stations in nearby hydrologically similar watersheds may be used. The discharge for such an ungaged stream may be determined by the transposition of records using a similar procedure. This procedure should be limited to sites that differ in area by no more than 50% from the area of the gage site. Following is an example using an exponent of 0.7.

In some streams, natural peak flows may decrease downstream because of significant floodplain storage or loss of flow in aquifer recharge zones. The designer is cautioned not to use the transposition approach in such cases.

TABLE 7-2 — Exponent For Hydrologic Regions

Watershed	Q_{25} (ft^3/s)	Area (acre)
Gaged Watershed A	61,800	471,673
Ungaged Watershed B	—	288,000

A: $61,800(288,000/471,673)^{0.7} = 49,706$ ft^3/s
B: $Q_{25} = 49,706$ ft^3/s

7.10 EXAMPLE PROBLEM — ANALYSIS OF STREAM GAGE DATA

EXAMPLE 1

Statistical method for computing flows: **Log-Pearson Type III**
Drainage basin area: Virgin River, near Hurricane, Utah

1. Obtain the Peak Stream flow for the Virgin River, near Hurricane, Utah, from the United State Geological Survey (USGS) Utah Surface Water data on the internet at: http://waterdata.usgs.gov/ut/nwis/peak?site_no=09408150&agency_cd=USGS&format=html . Verify sufficiency of continuous records (10 years min.). Screen the flow for possible outliers using the following equation: $Q_L = \underline{Q_L} + K_N S_L$, where:

Q_L is the log of the high and low outlier limit
$\underline{Q_L}$ is the mean of the log of the sample flows
K_N is the critical deviate (HEC 19, Table 24)
S_L is the standard deviation of sample Q_L

	Output formats
Washington County, Utah Hydrologic Unit Code 15010008 Latitude 37°10'20", Longitude 113°23'07" NAD27 Drainage area 1,499.00 square miles Gage datum 2,780.00 feet above sea level NGVD29	Table Graph Tab-separated file WATSTORE formatted file Reselect output format

Water Year	Date	Gage Height (feet)	Stream-flow (cfs)
1967	Dec. 6, 1966	17.34	20,100[5]
1968	Aug. 7, 1968	9.11	5,410[5]
1969	Jan. 25, 1969	14.29	12,800[5]
1970	Aug. 18, 1970	6.99	3,380[5]
1971	Aug. 21, 1971	9.00	6,250[5]
1972	Sep. 19, 1972	11.88	10,400[5]
1973	May 14, 1973	6.36	3,350[5]
1974	Apr. 2, 1974	3.67	1,090[5]
1975	Jul. 29, 1975	8.95	7,325[5]
1976	Jul. 29, 1976	4.96	2,064[5]
1977	Jul. 23, 1977	7.22	4,830[5]
1978	Mar. 5, 1978	16.28	18,700[5]
1979	Mar. 28, 1979	9.69	8,440[5]
1980	Sep. 10, 1980	11.33	10,910[5]
1981	Jul. 15, 1981	6.82	4,340[5]
1982	Sep. 27, 1982	8.62	6,810[5]
1983	Nov. 30, 1982	8.37	6,640[5]
1984	Jul. 23, 1984	7.83	5,660[5]
1985	Jul. 20, 1985	5.38	2,700[5]
1986	Nov. 30, 1985	4.48	1,560[5]
1987	Jul. 21, 1987	7.80	4,860[5]
1988	Nov. 6, 1987	9.43	7,310[5]
1989	Jan. 1, 1989		66,000[3,5]
1991	Mar. 1, 1991	13.62	747[5]
1992	Aug. 23, 1992	18.05	3,900[5]
1993	Feb. 20, 1993	17.69	4,870[5]
1994	Oct. 6, 1993	12.55	5,000[5]
1995	Mar. 6, 1995	15.63	7,020[5]
1996	Nov. 1, 1995	12.51	4,990[5]
1997	Jan. 3, 1997	12.71	5,120[5]
1998	Sep. 11, 1998	14.02	6,290[5]
1999	Aug. 30, 1999	12.68	5,110[5]
2000	Feb. 24, 2000	8.34	1,260[5]
2001	Oct. 30, 2000	8.88	1,810[5]

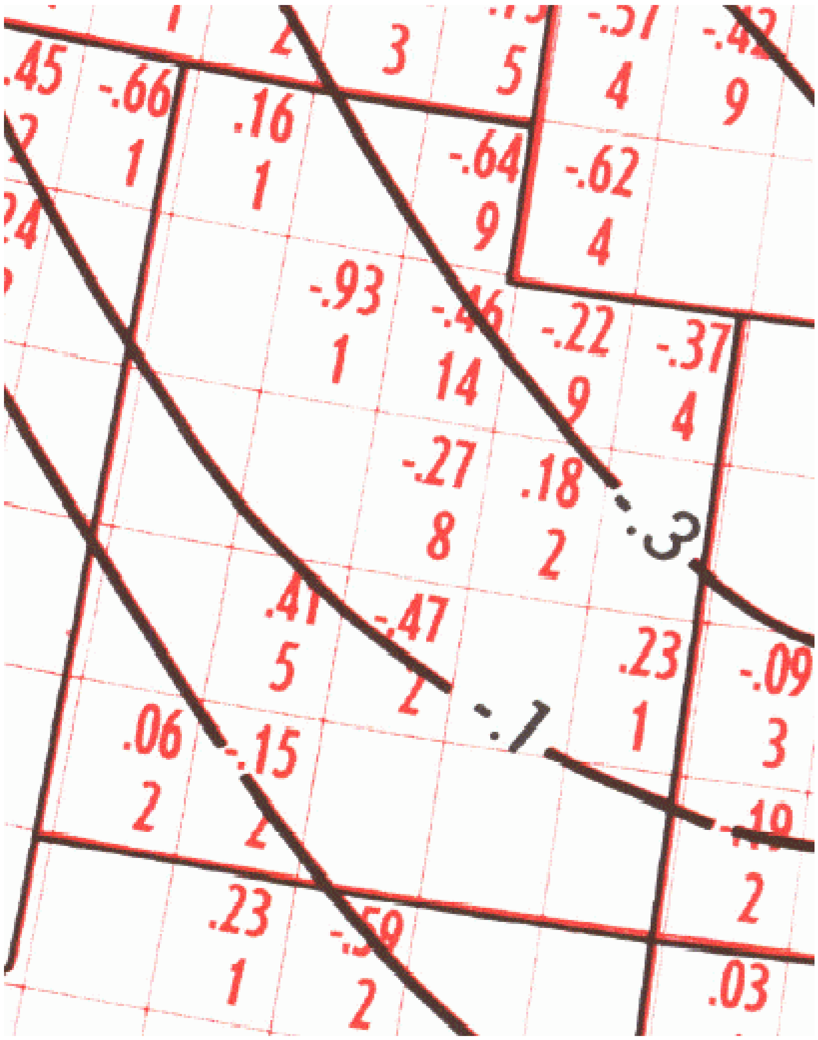
The log of the upper limit flow Q_L is equal to 4.70355 the upper limit flow is 50,530 cfs.
March 1, 1989 flow of 66,000 was the result of a dam failure and is an outlier.

2. Find the skew G using the following equation:

G = (N*Sum(X_L – Mean X_L)^3)/((N-1)*(N-2)*(SL^3))

G = -0.27

Check the calculated skew coefficient for reasonableness using the map below:



3. From the table below find the critical deviate K_N for the return period of interest.

Table 14. Cumulative Distribution Function for Log-Pearson Type III Distribution

Coef. of Skew	Exceedance Probability in %						
	50	20	10	4	2	1	0.2
	Corresponding Return Period in Years						
	2	5	10	25	50	100	500
3	-0.3955	0.4204	1.1801	2.2778	3.1519	4.0514	6.2051
2.8	-0.3835	0.4598	1.2101	2.2747	3.114	3.973	6.0186
2.6	-0.3685	0.4987	1.2377	2.2674	3.0712	3.8893	5.6282
2.4	-0.3506	0.5368	1.2624	2.2558	3.0233	3.8001	5.6282
2.2	-0.33	0.5738	1.2841	2.2397	2.9703	3.7054	5.4243
2	-0.3069	0.6094	1.3026	2.2189	2.912	3.6052	5.2146
1.8	-0.2815	0.6434	1.3176	2.1933	2.8485	3.4994	4.9994
1.6	-0.2542	0.6753	1.329	2.1629	2.7796	3.388	4.7788
1.4	-0.2254	0.7051	1.3367	2.1277	2.7056	3.2713	4.553
1.2	-0.1952	0.7326	1.3405	2.0876	2.6263	3.1494	4.3226
1	-0.164	0.7575	1.3404	2.0427	2.5421	3.0226	4.088
0.8	-0.132	0.7799	1.3364	1.9931	2.453	2.891	3.8498
0.6	-0.0995	0.7995	1.3285	1.939	2.3593	2.7551	3.6087
0.4	-0.0665	0.8164	1.3167	1.8804	2.2613	2.6154	3.3657
0.2	-0.0333	0.8304	1.3011	1.8176	2.1594	2.4723	3.1217
0	0	0.8416	1.2816	1.7507	2.0538	2.3264	2.8782
-0.2	0.0333	0.8499	1.2582	1.68	1.945	2.1784	2.6367
-0.4	0.0665	0.8551	1.2311	1.6057	1.8336	2.0293	2.3994
-0.6	0.0995	0.8572	1.2003	1.5283	1.7203	1.8803	2.1688
-0.8	0.132	0.8561	1.1657	1.4481	1.606	1.7327	1.9481
-1	0.164	0.8516	1.1276	1.3658	1.4919	1.5884	1.7406
-1.2	0.1952	0.8437	1.0861	1.2823	1.3793	1.4494	1.5502
-1.4	0.2254	0.8322	1.0414	1.1984	1.27	1.3182	1.3798
-1.6	0.2542	0.8172	0.9942	1.1157	1.1658	1.1968	1.2313
-1.8	0.2815	0.7986	0.945	1.0354	1.0686	1.0871	1.1047
-2	0.3069	0.7769	0.8946	0.9592	0.9798	0.99	0.998
-2.2	0.33	0.7521	0.8442	0.8881	0.9001	0.9052	0.9085
-2.4	0.3506	0.725	0.7947	0.8232	0.8296	0.832	0.8332
-2.6	0.3685	0.696	0.7471	0.7646	0.7678	0.7688	0.7692
-2.8	0.3835	0.666	0.7021	0.7123	0.7138	0.7142	0.7143
-3	0.3955	0.6357	0.6602	0.6659	0.6665	0.6667	0.6667
from WRC, 1981							

4. Using the calculated skew coefficients the critical deviate K_N is found in table 14: $Q_L = \underline{Q_L} + K_N S_L$

$S_L = 0.33$
$\underline{Q_L} = 3.68$

10 year $K_N = 1.2582$	$Q_L = 4.09$	Q = 12,388 cfs
50 year $K_N = 1.945$	$Q_L = 4.32$	Q = 20,868 cfs
100 year $K_N = 2.1784$	$Q_L = 4.40$	Q = 24,914 cfs

HYDRO in the HYDRAIN suite computer programs contains a template for computing Log Pearson Type III distribution. The suite can be downloaded from the FHWA web site.

7.11 UNIT HYDROGRAPH — GAGED DATA

7.11.1 Introduction

It is sometimes useful or necessary to estimate the runoff hydrograph associated with the peak discharge of a desired frequency. Several methods are available to develop a design hydrograph. At sites where gaged data are available, a unit hydrograph can be developed from corresponding rainfall and runoff data. Refer to Section 7.5 for a definition of a unit hydrograph.

A unit hydrograph is a hydrograph of the runoff resulting from a hypothetical storm that has a specified duration (e.g., 1 h) and that produces exactly one inch of runoff over the drainage area. When a unit hydrograph is shown with units of cubic feet per second, it is implied that the ordinates are cubic feet per second per inch of direct runoff. The unit hydrograph techniques described below can be used to approximate the rainfall-runoff response of a particular watershed and to predict the flood hydrograph that would result from a design storm of a desired frequency.

7.11.2 Application

The Department may use the unit hydrograph for design when sufficient gaged or synthesized data are not available to use an analysis of stream-gaged data procedure or when the watershed involves controls such as detention ponds or reservoirs.

7.11.3 Characteristics

To develop a unit hydrograph for a watershed, corresponding rainfall and runoff records must be available for the area. These records consist of flood discharge data at the desired site and corresponding rainfall data from the contributing watershed for the same flood event. Discharge data are plotted against time to produce a flood hydrograph. Rainfall records are usually obtained as rainfall mass curves that can be used to develop a graph of rainfall intensity over time (termed a rainfall hyetograph).

7.11.4 Procedure

Following is the procedure for construction of a unit hydrograph from rainfall and runoff data:

Step 1 DETERMINATION OF DIRECT RUNOFF HYDROGRAPH. The normal stream baseflow is subtracted from the flood hydrograph to produce a direct runoff hydrograph by drawing a straight line from the beginning of the rising portion of the hydrograph to a point directly under the peak at the same slope as the baseflow curve prior to the beginning of the flood hydrograph. A second straight line is then drawn from the end of the first line to connect to the recession limb of the hydrograph at a point where the baseflow is equal in magnitude to that where the hydrograph began. The direct runoff hydrograph is then determined by subtracting the baseflow from the flood hydrograph.

Step 2 DETERMINATION OF DIRECT RUNOFF VOLUME. The direct runoff is determined as the area under the direct runoff hydrograph. The volume is determined by dividing the time base of the hydrograph into convenient time increments, determining the average discharge for each time increment, and multiplying the length of the time increment by the average discharge to obtain the volume for that increment. This procedure is repeated for each time increment, and the incremental volumes are summed to obtain the total direct runoff volume. Another approach would be to planimeter the area under the runoff hydrograph. This volume, usually in cubic meters, is converted into millimeters by dividing by the total watershed area in square meters and converting this depth into millimeters.

Step 3 DETERMINATION OF UNIT HYDROGRAPH ORDINATES. The ordinates of the unit hydrograph are determined by dividing each ordinate of the flood hydrograph by the volume of direct runoff (in inches). The time base of the unit hydrograph will be the same, as well as the general shape. Only the magnitude of the discharge coordinates will be different. If the direct runoff volume is recomputed as described above, the total volume should equal one millimeter.

Step 4 DETERMINATION OF UNIT STORM DURATION. The storm that produced the unit hydrograph calculated above will have a specified duration. This duration is the length of time that the storm produced direct runoff. For example, if all of the rainfall during the first 15 min percolated into the ground, the first 15 min would not be part of the storm duration. The unit storm duration can be determined using the rainfall hyetograph and the direct runoff hydrograph. Because of the complexity of most rainfall records and the difficulty in accurately determining the amount of rainfall that is lost through infiltration and depression storage, two fairly simple adjustments are made to determine the total depth of rainfall that becomes direct runoff.

a. The time of the beginning of rainfall and the time of the beginning of the corresponding flood hydrograph are compared. Any rainfall occurring prior to the beginning of runoff is assumed to be lost due to initial abstractions and is subtracted from the storm.

b. The remaining volume of rainfall (the area under the hyetograph) is calculated and compared to the volume of direct runoff. The rainfall volume will be greater if infiltration and other losses have occurred during the runoff event. The Phi index method is used to subtract out these losses. The Phi index is defined as the rate of rainfall above which the rainfall volume equals the runoff volume. A straight line is drawn to indicate a constant loss rate; the position of this straight line is drawn such that the remaining volume under the hyetograph equals the volume of direct runoff computed from the direct runoff hydrograph.

The duration of the unit storm is the time during which the storm produces direct runoff. It is easily determined from the rainfall hyetograph once the two adjustments described above have been made. Once a unit hydrograph for a particular storm duration has been determined, unit hydrographs for other durations can be developed.

7.12 EXAMPLE PROBLEM — UNIT HYDROGRAPH

Figure 7-2 shows a rainfall hyetograph and the resulting direct runoff hydrograph (baseflow subtracted) produced by a particular storm over a watershed of 2405 hectares:

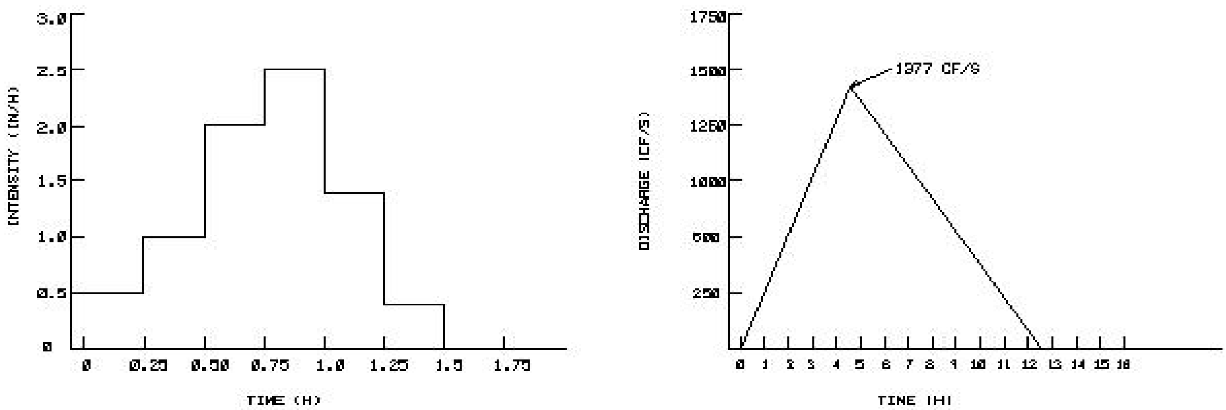


FIGURE 7-2 — Rainfall Hyetograph and Direct Runoff Hydrograph

A. <u>Determination of Direct Runoff Volume</u>

Volume = $(0.5)(1377\ ft^3/s)(3600\ s/h)(12.25 - 0.25)\ h$
= $29\ 743\ 200\ ft^3$

<u>Equivalent Depth of Runoff</u>

Depth = $(29\ 743\ 200\ ft^3/5943\ ac)(1\ ac/43\ 560\ ft^2)(12\ in/ft)$
= 1.4 in

B. <u>Determination of Unit Hydrograph Ordinates</u>

$Q_p/1.0\ in = (1377\ ft^3/s)/1.38\ in = 997.8\ ft^3/s$

Check Volume:

Volume = $(0.5)(997.8\ ft^3/s)(3600\ s/h)(12.25 - 0.25)\ h$
= $21\ 552\ 480\ ft^3$

Depth = $(21\ 552\ 480\ ft^3/5943\ ac)(1\ ac/43560\ ft^2)(12\ in/ft)$
= 1.0 in

C. <u>Determination of Storm Duration</u>

Because the direct runoff hydrograph begins at 0.25 h, the first increment of rainfall from 0 to 0.25 h satisfies the initial abstractions (interception, depression storage and infiltration). The remaining volume of rainfall is:

Volume (depth) = (0.25 h)(1.2 + 2.1 + 2.8 + 1.6 + 0.4) in/h
= 2 in

Because the direct runoff volume (depth) is 1.4 in, the remaining 0.6 (2 – 1.4) in must be lost due to infiltration during the runoff event. The phi index is adjusted by trial and error to yield the desired volume of excess rainfall. For a phi index = 0.5 in/h:

Excess rainfall = (0.6 + 1.5 + 2.2 + 1.0)(0.25) = 1.3 in

The duration of the storm producing this rainfall is 1 h.

7.13 RURAL REGRESSION EQUATIONS

7.13.1 Introduction

Regional regression equations are the most commonly accepted method for estimating peak flows at larger ungaged sites or sites with insufficient data for statistical evaluation of flood frequency. They are easy to use and provide consistent findings when applied by different hydraulics engineers (Reference *(8)*). For convenience to the designer, this *Manual* includes the USGS Regional Regression Equations for Utah on table 17-3 and 17-3a.

TABLE 7-3 — Regression Constants, Coefficients and Standard Errors

Region equations: Q_T, peak discharge in cubic feet per second; AREA, drainage area, in square miles; ELEV, mean basin elevation, in feet above sea level (NGVD of 1929)]	**Average standard error of prediction, %**	**Equivalent years of record**
Region 1 (For sites located at elevations greater than elevation threshold from Figure 7-3)		
$Q_2 = 0.124AREA^{0.845}PREC^{1.44}$	59	0.16
$Q_5 = 0.629AREA^{0.807}PREC^{1.12}$	52	.62
$Q_{10} = 1.43AREA^{0.786}PREC^{0.958}$	48	1.34
$Q_{25} = 3.08AREA^{0.768}PREC^{0.811}$	46	2.50
$Q_{50} = 4.75AREA^{0.758}PREC^{0.732}$	46	3.37
$Q_{100} = 6.78AREA^{0.750}PREC^{0.668}$	46	4.19
Region 3		
$Q_2 = 0.444AREA^{0.649}PREC^{1.15}$	86	0.29
$Q_5 = 1.21AREA^{0.639}PREC^{0.995}$	83	.49
$Q_{10} = 1.99AREA^{0.633}PREC^{0.924}$	80	.77
$Q_{25} = 3.37AREA^{0.627}PREC^{0.849}$	78	1.23
$Q_{50} = 4.70AREA^{0.625}PREC^{0.802}$	77	1.57
$Q_{100} = 6.42AREA^{0.621}PREC^{0.757}$	78	1.92
Region 4		
$Q_2 = 0.0405AREA^{0.701}(ELEV/1,000)^{2.91}$	64	0.39
$Q_5 = 0.408AREA^{0.683}(ELEV/1,000)^{2.05}$	57	95
$Q_{10} = 1.26AREA^{0.674}(ELEV/1,000)^{1.64}$	53	1.76
$Q_{25} = 3.74AREA^{0.667}(ELEV/1,000)^{1.24}$	51	3.02
$Q_{50} = 7.04AREA^{0.664}(ELEV/1,000)^{1.02}$	52	3.89
$Q_{100} = 11.8AREA^{0.662}(ELEV/1,000)^{0.835}$	53	4.65

Region 7		
$Q_2 = 0.0150AREA^{0.697}(ELEV/1,000)^{3.16}$	56	0.25
$Q_5 = 0.306AREA^{0.590}(ELEV/1,000)^{2.22}$	45	1.56
$Q_{10} = 1.25AREA^{0.526}(ELEV/1,000)^{1.83}$	45	3.07
$Q_{25} = 122AREA^{0.440}$	49	4.60
$Q_{50} = 183AREA^{0.390}$	53	5.27
$Q_{100} = 264AREA^{0.344}$	59	5.68
Region 8		
$Q_2 = 598AREA^{0.501}(ELEV/1,000)^{-1.02}$	72	0.37
$Q_5 = 2,620AREA^{0.449}(ELEV/1,000)^{-1.28}$	62	1.35
$Q_{10} = 5,310AREA^{0.425}(ELEV/1,000)^{-1.40}$	57	2.88
$Q_{25} = 10,500AREA^{0.403}(ELEV/1,000)^{-1.49}$	54	5.45
$Q_{50} = 16,000AREA^{0.390}(ELEV/1,000)^{-1.54}$	53	7.45
$Q_{100} = 23,300AREA^{0.377}(ELEV/1,000)^{-1.59}$	53	9.28
Region 9		
$Q_2 = 0.0204AREA^{0.606}(ELEV/1,000)^{3.5}$	68	0.14
$Q_5 = 0.181AREA^{0.515}(ELEV/1,000)^{2.9}$	55	.77
$Q_{10} = 1.18AREA^{0.488}(ELEV/1,000)^{2.2}$	52	1.70
$Q_{25} = 18.2AREA^{0.465}(ELEV/1,000)^{1.1}$	53	2.81
$Q_{50} = 248AREA^{0.449}$	57	3.36
$Q_{100} = 292AREA^{0.444}$	59	3.94

The regression equation for region 6, shown on Table 7-3 a was developed using an iterative regression method (Hjalmarson and Thomas, 1992) and a modified form of the station year statistical analysis method (Fuller, 1914). The average standard error of regression is an estimate of the predictive accuracy of these regression equations and is determined by a direct sampling method.

TABLE 7-3 a — Regression Constants, Coefficients and Standard Errors for Region 6

Region 6*		
Regression equations: Q_T, peak discharge, in cubic feet per second; AREA, drainage area, in square miles; ELEV, mean basin elevation, in feet above sea level (NGVD of 1929)	**Standard error of regression, in log units**	**Equivalent years of record**
$Q_2 = 0$	--	--
$Q_5 = 32AREA^{0.80}(ELEV/1,000)^{-0.66}$	1.47	0.233
$Q_{10} = 590AREA^{0.62}(ELEV/1,000)^{-1.6}$	1.12	.748
$Q_{25} = 3,200AREA^{0.62}(ELEV/1,000)^{-2.1}$	.796	2.52
$Q_{50} = 5,300AREA^{0.64}(ELEV/1,000)^{-2.1}$	1.10	1.75
$Q_{100} = 20,000AREA^{0.51}(ELEV/1,000)^{-2.3}$	1.84	.794

*(modified from Thomas and others, 1997)

Estimated average standard error of regression for these equations includes much of the within-station residual variance and therefore is not comparable to standard error of estimate from an ordinary-least-squares regression.

Appendix 7.E includes a map with the average annual precepitation in Utah.

7.13.2 Regression Equations and Methods

Regression equations use stream gage data from hydraulically homogeneous catchment areas for establishing flows at similar ungaged sites. Care must be exercised when using regression equations:

- Determine if the design site lies on or near a regional boundary,
- Collect all available historical flood data *(20)*,
- Use the gathered data to interpret the discharge values.

Where the findings from these regression equations are found to be contrary to historical observations or other methods, the channel geometry characteristics method shall be used to resolve the differences. To verify the estimated runoffs the designer should compare them to the maximum stream historical flows. The Utah Department of Natural Resources published the Summary Of Maximum Stream Discharges In Utah Streams (Technical Bulletin No. 21 *(20)*).

Figure 7-3 shows the hydrologic regional boundaries for the state of Utah as outlined by the USGS (Reference *(17)*).

If problems related to hydrologic boundaries may occur in selecting the appropriate regression equation. For example if the watershed of interest may lie partly within two or more hydrologic regions then follow the protocols detailed in the USGS reference. A problem may occur when a watershed lies totally within a hydrologic region but close to a hydrologic region boundary.

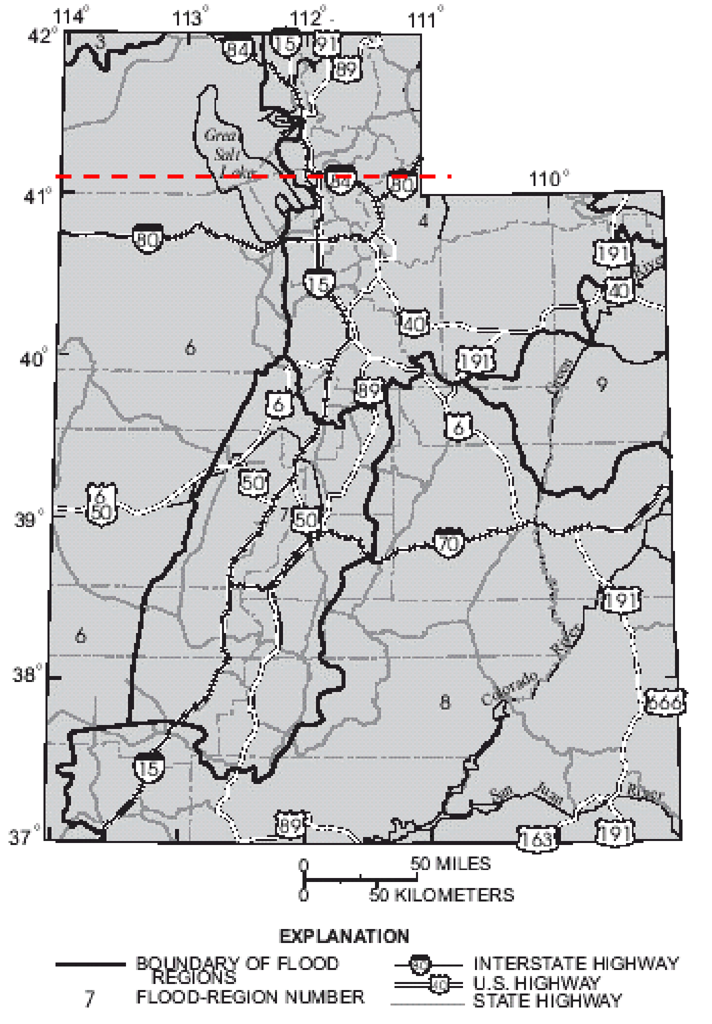
7.13.3 Procedure

The following procedure shall be used in applying the USGS regression equations:

WATERSHEDS HAVING NO BOUNDARY OR MIXED-POPULATION COMPLEXITIES. Collect the data from such items as maps, field visits, surveys and aerial photos and the enclosed figures and apply the following equations. Hydrologic regions are defined on Figure 7-3, and the regression constants, coefficients and standard errors are in Table 7-3. With sensitive sites, try other methods to see if the findings are reasonable. Compute a flood-frequency curve to include the 2-, 5-, 10-, 25-, 50- and 100-yr events at a minimum.

WATERSHEDS TRANSCENDING HYDROLOGIC REGION BOUNDARIES. Where a watershed transcends hydrologic boundaries, the total discharge at a site for the foregoing recurrence intervals shall be determined by the USGS. The foregoing regression equations shall be applied using the above procedure and the following equations. Following are equations for sites in transition zones:

When the drainage area of the site of interest is in more than one of the regions 3, 4, 6, 7, 8, or 9, a weighted estimate of the peak discharge should be computed. The equations for the appropriate regions should be applied independently by using basinwide estimates of the required explanatory variables. The weighted estimate is then computed by multiplying each regional estimate against the fraction of the drainage area in that region and summing the products. The NFF Program provides an algorithm for this computation.



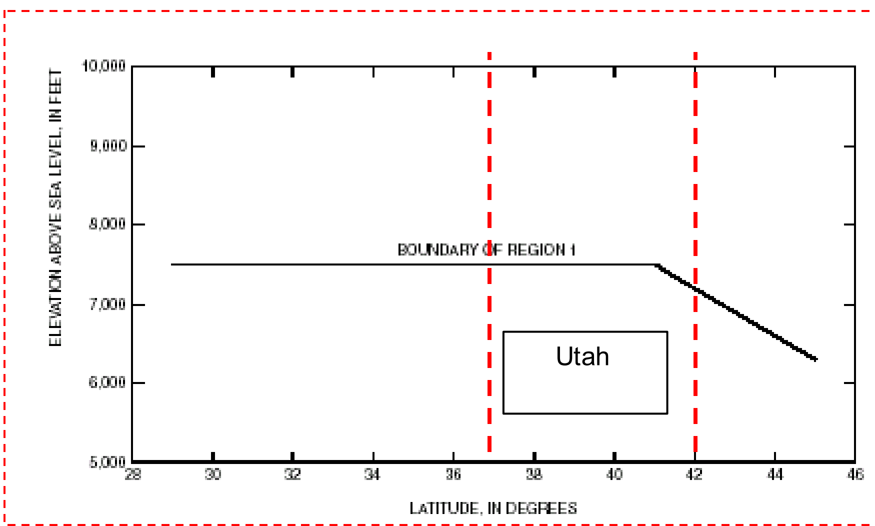


FIGURE 7-3 — Utah Hydrologic Regions

When the elevation of the stream site of interest is between 6,800 and 7,500 feet, a weighted estimate of the peak discharge should be computed by using the equations for region 1 and the other regions in which the basin is located. The applicable equations are each applied by using basinwide estimates of the required explanatory variables, and the region estimates are weighted as a function of elevation as follows:

$$Q_T(W) = Q_T(u) \cdot \frac{7,500 - E}{700} + Q_T(Region1) \cdot \left(1 - \frac{7,500 - E}{700}\right)$$

$Q_T(W)$ is the weighted of peak-discharge estimate for the recurrence interval T at the site of interest,

$Q_T(u)$ is the estimate of peak discharge using the equations for regions 3, 4, 6, 7, 8 or 9 as appropriate,

Q_T *(Region1)* is the estimate of the peak discharge using the equations for region 1, and

E is the elevation of the stream site of interest.

The NFF Program does not provide an algorithm for this weighting computation. Thomas and others (1997) summarized the basin characteristics, the estimates of peak discharge, and the weighted estimates of peak discharge for most of the 1,323 sites used in the study, including 212 sites in Utah (Reference *(17)*).

WATERSHEDS CLOSE TO HYDROLOGIC BOUNDARIES. Where a watershed is close to a hydrologic boundary, the appropriate hydrologic region shall be selected based on the findings from the field inspection. Sometimes, a specific variable (e.g., elevation, rainfall isohyetals) will better define a hydrologic region boundary. To facilitate the identification of hydrologic boundaries, the following procedure shall be used. The findings from a channel geometry analysis may have an important role in identifying the boundaries between hydrologic regions.

WATERSHEDS SUBJECT TO MIXED-POPULATION FLOODS. Where two or more flood populations are expected to comprise the flood of interest, the total discharge at a site for the foregoing recurrence intervals shall be determined by USGS procedure. The following equations shall be used as indicated.

TRANSFERRING GAGED DATA. Gaged data may be transferred to an ungaged site of interest, provided that such data are nearby.

i.e., within the same hydrologic region, and there are no major tributaries or diversions between the gage and the site of interest. These procedures make use of the constants obtained in developing the regression equations. These procedures are as follows:

Thomas and others (1997) showed how the weighted estimate of peak discharge can be used to improve estimates of peak discharge of an ungaged site on the same stream that has a drainage area that is between 50 and 150 percent of the drainage area of the gaged site. The weighted estimate is computed as:

$$Q_T(u) = Q_T(W) \cdot \left(\frac{Area_{ungaged}}{Area_{gaged}}\right)^b ,$$

where

$Q_T(u)$ is the weighted peak-discharge estimate for the recurrence interval T at the ungaged site,

$Q_T(W)$ is the weighted estimate of peak discharge at the gaged site,

$AREA_{ungaged}$ and $AREA_{gaged}$ are the drainage areas of the ungaged and gaged sites, respectively, and

b is an exponent for each region as follows:

Region	Exponents
1	0.8
3	.7
4	.7
6	.6
7	.5
8	.4
9	.5

The adjustment to the weighted estimate of peak discharge at the gaged site can be used when the drainage area at the ungaged site is within 50 to 150 percent of the drainage area of the gaged site. Otherwise, the estimate at the ungaged site should be based on the appropriate regression equation only (Reference *(17)*).

7.13.4 National Flood Frequency Program

USGS, in cooperation with FHWA and FEMA, has compiled all current Statewide and urban-area regression equations into the National Flood Frequency computer program. This program includes regression equations for estimating flood-peak discharges and techniques for estimating a typical flood hydrograph for a given recurrence interval peak discharge for unregulated rural and urban watersheds. The regression equations for estimating flood-peak discharges for rural, unregulated watersheds have been developed for every State and the Commonwealth of Puerto Rico. Regression equations for estimating urban flood-peak discharges for several urban areas in at least 13 states are also available from the program. For a complete report, consult Reference *(17).*

7.14 EXAMPLE PROBLEM — RURAL DISCHARGES FROM REGRESSION EQUATIONS

Method for computing flows: **USGS Rural Regression Equations**
Drainage basin area: Virgin River, near Hurricane, Utah (Same as Example 1)

1. Use WMS (Watershed Modeling System) to define the basin boundaries, get the basin area and average basin elevation. Drainage area: 1499 Square Miles; Average Basin Elevation: 5375 feet.
2. From the map on figure 7.3 find the hydrologic region in which the basin is. The upper basin of the Virgin River is in region 8.
3. Use the given equations to calculate the run-offs for the various return periods:

- $Q_2 = 598\ 1499^{0.501}(5375/1{,}000)^{-1.02} = 4{,}196$ cfs
- $Q_5 = 2{,}620\ 1499^{0.449}(5375/1{,}000)^{-1.28} = 8{,}116$ cfs
- $Q_{10} = 5{,}310\ 1499^{0.425}(5375/1{,}000)^{-1.40} = 11{,}279$ cfs
- $Q_{25} = 10{,}500\ 1499^{0.403}(5375/1{,}000)^{-1.49} = 16{,}322$ cfs
- $Q_{50} = 16{,}000\ 1499^{0.390}(5375/1{,}000)^{-1.54} = 20{,}792$ cfs
- $Q_{100} = 23{,}300\ 1499^{0.377}(5375/1{,}000)^{-1.59} = 25{,}312$ cfs

Or use the USGS-NFF program for Windows to apply the same equations:

National Flood Frequency Program
Version 3.0
Based on Water-Resources Investigations Report 02-4168
Equations from database NFFv3.mdb
Updated by cdperl 11/19/02 at 11:42:41 AM Added from WRIR 96-4176
Equations for Utah developed using English units

Site: Virgin River, Utah
User:
Date: Tuesday, January 21, 2003 10:36 AM

Rural Estimate: Rural 1
  Basin Drainage Area: 1500 mi2
  1 Region
  Region: Four_Corners_Region_8
    Drainage_Area = 1500 mi2
    Mean_Basin_Elevation = 5380 ft
  Crippen & Bue Region 14

Rural Estimate: Rural 1 (weighted)
  Basin Drainage Area: 1500 mi2
  1 Region
  Region: Four_Corners_Region_8
    Drainage_Area = 1500 mi2
    Mean_Basin_Elevation = 5380 ft
  Crippen & Bue Region 14

Flood Peak Discharges, in cubic feet per second

Estimate	Recurrence Interval, yrs	Peak, cfs	Standard Error, %	Equivalent Years
Rural 1	2	4200	72	0.4
	5	8120	62	1.4
	10	11300	57	2.9
	25	16300	54	5.5
	50	20800	53	7.5
	100	25300	53	9.3
	500	38900		

maximum: 74100 (for C&B region 14)

Estimate	Recurrence Interval, yrs	Peak, cfs	Standard Error, %	Equivalent Years
Rural 1 (weighted)	2	4200	72	0.4
	5	8120	62	1.4
	10	11300	57	2.9
	25	16300	54	5.5
	50	20800	53	7.5
	100	25300	53	9.3
	500	38900		

maximum: 74100 (for C&B region 14)

7.15 URBAN REGRESSION EQUATIONS

7.15.1 Introduction

Regression equations developed by USGS *(12)* as part of a nationwide project can be used to estimate peak runoff for urban watershed conditions. The regression equations for urban watersheds compiled in the National Flood Frequency Program are addressed in Section 7.13.8.

Reference *(12)* provides two sets of seven-parameter equations and a third set based on three parameters. The equations account for regional runoff variations through the use of the equivalent rural peak runoff rate (RQ). The equations adjust RQ to an urban condition using the basin development factor (BDF) and the percentage of impervious area (IA). Some areas have had good success using a three-parameter equation. If local equations are available, they should be included.

7.15.2 Application

The hydraulics engineer may apply these urban equations for the final design of bridges, culverts and similar structures where such structures are not an integral part of a storm drain system and provided that the contributing watershed either is, or expected to become, predominately 75% urban in nature.

7.15.3 Typical Equations and Characteristics

The nationwide equations for urban conditions (see Table 7-4) take the following general form:

TABLE 7-4 — USGS Nationwide Regression Equations For Urban Conditions

Standard Peak Runoff Equation	R^2	Error %
$UQ_2 = 2.35A^{0.41}\ SL^{0.17}\ (RI2 + 3)^{2.04}\ (ST + 8)^{-0.65}\ (13 - BDF)^{-0.32}\ IA^{0.15}\ RQ_2^{0.47}$	0.93	38
$UQ_5 = 2.70A^{0.35}\ SL^{0.16}\ (RI2 + 3)^{1.86}\ (ST + 8)^{-0.59}\ (13 - BDF)^{-0.31}\ IA^{0.11}\ RQ_5^{0.54}$	0.93	38
$UQ_{10} = 2.99A^{0.32}\ SL^{0.15}\ (RI2 + 3)^{1.75}\ (ST + 8)^{-0.57}\ (13 - BDF)^{-0.30}\ IA^{0.09}\ RQ_{10}^{0.58}$	0.93	38
$UQ_{25} = 2.78A^{0.31}\ SL^{0.15}\ (RI2 + 3)^{1.76}\ (ST + 8)^{-0.55}\ (13 - BDF)^{-0.29}\ IA^{0.07}\ RQ_{25}^{0.60}$	0.93	40
$UQ_{50} = 2.67A^{0.29}\ SL^{0.15}\ (RI2 + 3)^{1.74}\ (ST + 8)^{-0.53}\ (13 - BDF)^{-0.28}\ IA^{0.06}\ RQ_{50}^{0.62}$	0.92	42
$UQ_{100} = 2.50A^{0.29}\ SL^{0.15}\ (RI2 + 3)^{1.76}\ (ST + 8)^{-0.52}\ (13 - BDF)^{-0.28}\ IA^{0.06}\ RQ_{100}^{0.63}$	0.92	44
$UQ_{500} = 2.27A^{0.29}\ SL^{0.16}\ (RI2 + 3)^{1.86}\ (ST + 8)^{-0.54}\ (13 - BDF)^{-0.27}\ IA^{0.05}\ RQ_{500}^{0.63}$	0.90	49

$$UQ_T = (\text{Coef})\ CA^{b1}\ SL^{b2}\ (RI2 + 3)^{b3}\ (ST + 8)^{b4}\ (13 - BDF)^{b5}\ IA^{b6}\ RQ_T^{b7} \quad (7.8)$$

where:

Coef = Coefficient – See USGS Web Site for equation

UQ_T = peak discharge, in ft^3/s, for the urban watershed for recurrence interval T

C = regression constant

A = contributing drainage area, in mi^2

SL = main channel slope, in ft/ft, measured between points that are 10% and 85% of the main channel length upstream from the study site (for sites where SL is greater than 0.0133 ft/ft; 0.0133 ft/ft is used in the equations)

RI2 = rainfall intensity, in inches, for the 2-h, 2-yr occurrence.

ST = basin storage, the percentage of the drainage basin occupied by lakes, reservoirs, swamps and wetlands (channel storage of a temporary nature, resulting from detention ponds or roadway embankment, is not included in the computation of ST)

BDF = basin development factor, an index of the prevalence of the drainage aspects of (a) storm drains, (b) channel improvements, (c) impervious channel linings, and (d) curb and gutter streets. The range of BDF is 0 to 12. A value of zero for BDF indicates that the above drainage aspects are not prevalent but does not necessarily mean the basin is non-urban. A value of 12 indicates full development of the drainage aspects throughout the basin.

IA = percentage of the drainage basin occupied by impervious surfaces (e.g., houses, buildings, streets, parking lots)

RQ_T = peak discharge, in ft^3/s, for an equivalent rural drainage basin in the same hydrologic area as the urban basin and for recurrence interval T b1, b2, b3 = regression exponents

7.15.4 Procedure

The procedure is not intended to require precise measurements. A certain amount of subjectivity will be involved, and field checking should be performed to obtain the best estimate. The BDF is the sum of the assigned codes; therefore, with three subbasins per basin, and four drainage aspects to which codes are assigned in each subbasin, the maximum value for a fully developed drainage system would be 12. Conversely, a totally undeveloped drainage system would receive a BDF of zero. This rating does not necessarily mean that the basin is unaffected by urbanization. In fact, a basin could be partially urbanized, have some impervious area and have some improvement of secondary tributaries, and still have an assigned BDF of zero. The following steps are used to apply the nationwide Equations:

1. Use the USGS regression equations for natural flow conditions to estimate the peak runoff rate for an equivalent rural drainage basin (RQ_T).

2. Determine input parameters for urban conditions, including main channel slope (SL), rainfall intensity (RI2), basin storage (ST), basin development factor (BDF) and impervious area (IA). The BDF should be determined from drainage maps and by field inspection of the watershed.

3. Calculate peak runoff rates for desired return periods.

The basin is first divided into thirds (see Figure 7-4) and, within each third, four aspects of the drainage system are evaluated and assigned a code as follows.

7.15.4.1 Channel Improvements

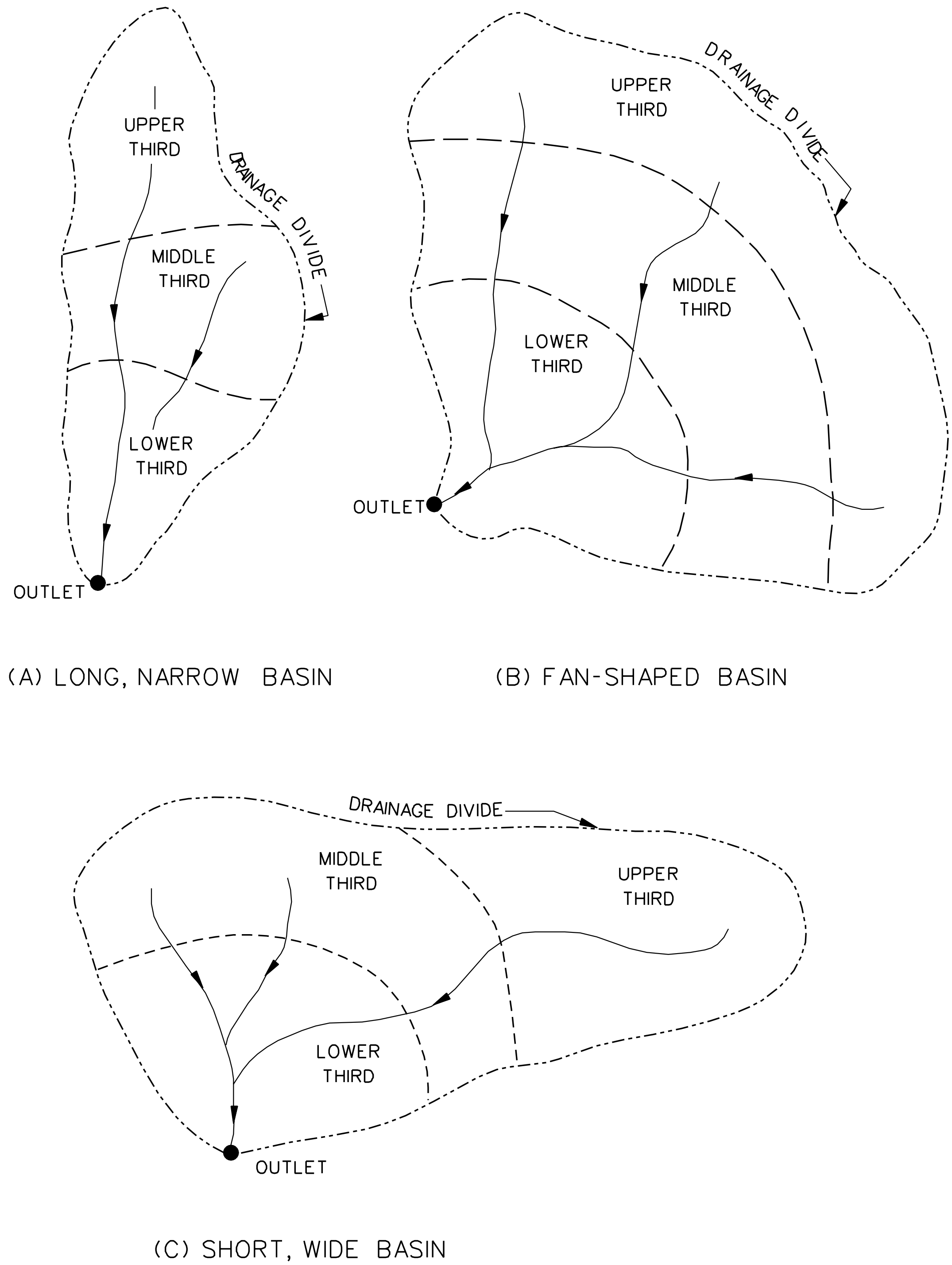
If channel improvements (e.g., straightening, enlarging, deepening, clearing) are prevalent for the main drainage channels and principal tributaries (those that drain directly into the main channel), then a code of one is assigned. To be considered prevalent, at least 50% of the main drainage channels and principal tributaries must be improved to some degree over natural conditions. If channel improvements are not prevalent, then a code of zero is assigned.

7.15.4.2 Channel Linings

If more than 50% of the length of the main drainage and principal tributaries has been lined with an impervious material (e.g., concrete), then a code of One is assigned to this aspect. If less than 50% of these channels are lined, then a code of Zero is assigned. The presence of channel linings is a good indication that channel improvements have been performed and signifies a more highly developed drainage system.

7.15.4.3 Storm Drains

Storm drains are enclosed drainage structures (usually pipes) frequently used on the secondary tributaries that receive drainage directly from streets or parking lots. Many of these drains empty into open channels; in some basins, however, they empty into channels enclosed as box or pipe culverts. Where more than 50% of the secondary tributaries within a subbasin consists of storm drains, a code of One is assigned to this aspect; if less than 50%, then a code of Zero. Note that, if 50% or more of the main drainage channels and principal tributaries are enclosed, the aspects of channel improvements and channel linings would also be assigned a code of One.



Schematic of typical drainage basin shapes and subdivision into basin thirds. Note that stream-channel distances within any given third of a basin in the examples are approximately equal but, between basin thirds, the distances are not equal to compensate for relative basin width of the thirds.

FIGURE 7-4 — Basin Subdivision

7.15.4.4 Curb and Gutter

If more than 50% of a subbasin is urbanized, and if more than 50% of the streets and highways in the subbasin are constructed with curbs and gutters, then a code of One would be assigned to this aspect. Otherwise, it would receive a code of Zero. Drainage from curb and gutter streets frequently empties into storm drains.

7.16 EXAMPLE PROBLEM — URBAN REGRESSION EQUATIONS

Method for computing flows: **USGS Urban Equations**
Drainage basin area: Provo, Utah – at 100 East and 300 West (South East Provo)

1. Use WMS (Watershed Modeling System) to define the basin boundaries, get the basin area and average basin elevation. Drainage area: 2.47 Square Miles; Average Basin Elevation: 4,722 feet.

2. Find the main channel slope (SL): SL = 0.0132 ft/ft

3. Find the rainfall intensity for the 2-hr, 2-yr occurrence (RI2): RI2 = 0.4 in

4. Find the storage (ST) if any within the drainage basin: ST = 0

5. Find the basin BDF by following the procedure outlined above: BDF = 8

6. Estimate the percent of the area occupied by impervious surfaces, IA = 55%

7. Find the peak discharge from equivalent rural basin(s) in the area using the USGS rural regression equation for each return year:

RQ_2 = 6.9 cfs; RQ_5 = 18.2 cfs; RQ_{10} = 29.5 cfs; RQ_{25} = 46.9 cfs; RQ_{50} = 62.5 cfs; RQ_{100} = 78.5 cfs;

8. Use the given equations to calculate the run-offs for the various return periods:

UQ_2 = 13.9cfs; UQ_5 = 24 cfs; UQ_{10} = 32.8 cfs; UQ_{25} = 44.1 cfs; UQ_{50} = 53.3 cfs; UQ_{100} = 62.8 cfs.

7.17 TIME OF CONCENTRATION

7.17.1 Overview

The time of concentration, which is denoted as t_c, is defined as the time required for a particle of water to flow from the hydraulically most distant point in the watershed to the outlet or design point. Factors that affect the time of concentration are the length of flow, the slope of the flow path and the roughness of the flow path. For flow at the upper reaches of a watershed, rainfall characteristics, most notably the intensity, may also influence the velocity of the runoff.

Various methods can be used to estimate the time of concentration of a watershed. When selecting a method to use in design, it is important to select a method that is appropriate for the flow path. Some estimation methods were designed and calibrated to be used for an entire watershed; the NRCS lag formula is an example of this method. These methods have t_c as the dependent variable. Other methods are intended for one segment of the principal flow path and produce a flow velocity that can be used with the length of that segment of the flow path to compute the travel time on that segment. With this method, the time of concentration equals the sum of the travel times on each segment of the principal flow path.

In classifying these methods so that the proper method can be selected, it is useful to describe the segments of flow paths. Sheet flow occurs in the upper reaches of a watershed. Such flow occurs over short distances and at shallow depths prior to the point where topography and surface characteristics cause the flow to concentrate in rills and swales. The depth of such flow is usually 0.5 in to 1.2 in or less. Concentrated flow is runoff that occurs in rills and swales and has depths on the order of 1.5 in to 4 in. Part of the principal flow path may include pipes or small streams. The travel time through these segments would be computed separately. Velocities in open channels are usually determined assuming bank-full depths.

7.17.2 Sheet-Flow Travel Time

Sheet flow is a shallow mass of runoff on a plane surface with the depth uniform across the sloping surface. Typically, flow depths will not exceed 2 in. Such flow occurs over relatively short distances, rarely more than about 300 ft, but most likely less than 100 ft. Sheet flow rates are commonly estimated using a version of the kinematic wave equation. The original form of the kinematic wave time of concentration is:

$$t_c = \frac{0.933}{I^{0.4}}\left(\frac{nL}{\sqrt{S}}\right)^{0.6} \qquad (7.9)$$

in which t_c is the time of concentration (minutes), n is the roughness coefficient ($ft^{1/3}/s$), L is the flow length (ft), I is the rainfall intensity (in/h) for a storm that has a return period T and duration of t_c (minutes), and S is the slope of the surface in ft/ft. Values of n can be obtained from Table 7-5.

TABLE 7-5 — Roughness Coefficients (Manning's n) For Sheet Flow

Surface Description	n[1]
Smooth surfaces (concrete, asphalt, gravel, bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grasses:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1] The n values are a composite of information compiled in Reference *(15)*.

[2] Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass and native grass mixtures.

[3] When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Some hydrologic design methods, such as the Rational method, assume that the storm duration equals the time of concentration. Thus, the time of concentration is entered into the IDF curve to find the design intensity. However, for Equation 7.9, i depends on t_c and t_c is not initially known. Therefore, the computation of t_c is an iterative process. An initial estimate of t_c is assumed and used to obtain i from the intensity-duration-frequency curve for the locality. The t_c is computed from Equation 7.9 and used to check the initial value of i. If they are not the same, then the process is repeated until two successive t_c estimates are the same.

7.17.3 Velocity Method

The velocity method can be used to estimate travel times for sheet flow, shallow concentrated flow, pipe flow or channel flow. It is based on the concept that the travel time (T_t) for a flow segment is a function of the length of flow (L) and the velocity (V):

$$T_t = \frac{L}{60V} \tag{7.10}$$

in which T_t, L and V have units of minutes, meters and meters/second, respectively. The travel time is computed for the principal flow path. Where the principal flow path consists of segments that have different slopes or land covers, the principal flow path should be divided into segments and Equation 7.10 used for each flow segment. The time of concentration is then the sum of travel times:

$$t_c = \sum_{i=1}^{k} T_{ti} = \sum_{i=1}^{k} \left(\frac{L_i}{60V_i} \right) \tag{7.11}$$

in which k is the number of segments and the subscript i refers to the flow segment.

The velocity of Equation 7.10 is a function of the type of flow (overland, sheet, rill and gully flow, channel flow, pipe flow), the roughness of the flow path, and the slope of the flow path. Some methods also include a rainfall index such as the 2-yr, 24-hour rainfall depth. A number of methods have been developed for estimating the velocity.

After short distances, sheet flow tends to concentrate in rills and then gullies of increasing proportions. Such flow is usually referred to as shallow concentrated flow. The velocity of such flow can be estimated using an empirical relationship between the velocity and the slope:

$$V = kS^{0.5} \tag{7.12}$$

in which V is the velocity (ft/s) and S is the slope (%). The value of k is a function of the land cover, with values for selected land covers given in Table 7-6.

7.17.4 Open Channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs or where blue lines (indicating streams) appear on USGS quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bankfull condition.

TABLE 7-6 Intercept Coefficients for Velocity vs. Slope Relationship of Equation 7.12

K	Land Cover/Flow Regime
0.25	Forest with heavy ground litter; hay meadow (overland flow)
0.5	Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)
0.7	Short grass pasture (overland flow)
0.9	Cultivated straight row (overland flow)
1.0	Nearly bare and untilled (overland flow); alluvial fans in western mountain regions
1.5	Grassed waterway (shallow concentrated flow)
1.61	Unpaved (shallow concentrated flow)
2.0	Paved area (shallow concentrated flow); small upland gullies

Manning's equation is:

$$V = \frac{1.49\ R^{2/3}\ S^{1/2}}{n} \tag{7.13}$$

where:
- V = average velocity, ft/s
- R = hydraulic radius, ft (equal to A/WP)
- A = cross sectional flow area, ft^2
- WP = wetted perimeter, ft
- S = slope of the hydraulic grade line, ft/ft
- n = Manning's roughness coefficient

After average velocity is computed using Equation 7.13, T_t for the channel segment can be estimated using Equation 7.10.

7.17.5 Reservoir or Lake

Sometimes, it is necessary to compute a T_c for a watershed having a relatively large body of water in the flow path. In such cases, T_c is computed to the upstream end of the lake or reservoir and, for the body of water, the travel time is computed using the Equation:

$$V_w = (gD_m)^{0.5} \tag{7.14}$$

where: V_w = the wave velocity across the water, ft/s
g = 32.2 ft/s^2
D_m = mean depth of lake or reservoir, ft

Generally, V_w will be high (8 to 30 ft/s).

One must not overlook the fact that Equation 7.14 only provides for estimating travel time across the lake and for the inflow hydrograph to the lake's outlet. It does not account for the travel time involved with the passage of the inflow hydrograph through spillway storage and the reservoir or lake outlet. This time is generally much longer and is added to the travel time across the lake. The travel time through lake storage and its outlet can be determined by the storage routing procedures in the Storage Chapter. Equation 7.14 can be used for swamps with considerable open water but, where the vegetation or debris is relatively thick (less than about 25% open water), Manning's equation is more appropriate. For additional discussion of Equation 7.14, see Reference *(11),* page 142.

7.18 RATIONAL METHOD

7.18.1 Introduction

The Rational method is recommended for estimating the design storm peak runoff for areas as large as 300 acres. This method, while first introduced in 1889, is still used in many engineering offices in the United States. Even though it has frequently come under criticism for its simplistic approach, no other drainage design method has received such widespread use.

7.18.2 Application

Some precautions should be considered when applying the Rational method.

- The first step in applying the Rational method is to obtain a good topographic map and define the boundaries of the drainage area in question. A field inspection of the area should also be made to determine if the natural drainage divides have been altered.

- In determining surface characteristics for the drainage area, thought should be given to future changes in land use that might occur during the service life of the proposed facility that could result in an inadequate drainage system. Also, the effects of upstream detention facilities may be considered.

- Restrictions to the natural flow (e.g., highway crossings and dams that exist in the drainage area) should be investigated to see how they affect the design flows.

- The charts, graphs and tables included in this Section are not intended to replace reasonable and prudent engineering judgment that should permeate each step in the design process.

7.18.3 Characteristics

Characteristics of the Rational method which limit its use to 300 acres include:

(1) The rate of runoff resulting from any rainfall intensity is a maximum when the rainfall intensity lasts as long or longer than the time of concentration. That is, the entire drainage area does not contribute to the peak discharge until the time of concentration has elapsed.

This assumption limits the size of the drainage basin that can be evaluated by the Rational method. For large drainage areas, the time of concentration can be so large that constant rainfall intensities for such long periods do not occur and shorter more intense rainfalls can produce larger peak flows. Further, in semi-arid and arid regions, storm cells are relatively small with extreme intensity variations thus making the Rational method inappropriate for watersheds greater than approximately 300 acres.

(2) The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration.

Frequencies of peak discharges depend on rainfall frequencies, antecedent moisture conditions in the watershed and the response characteristics of the drainage system. For small and largely impervious areas, rainfall frequency is the dominant factor. For larger drainage basins, the response characteristics control. For drainage areas with few impervious surfaces (less urban development), antecedent moisture conditions usually govern, especially for rainfall events with a return period of ten years or less.

(3) The fraction of rainfall that becomes runoff (C) is independent of rainfall intensity or volume.

The assumption is reasonable for impervious areas (e.g., streets, rooftops, parking lots). For pervious areas, the fraction of runoff varies with rainfall intensity and the accumulated volume of rainfall. Thus, the art necessary for application of the Rational method involves the selection of a coefficient that is appropriate for the storm, soil and land-use conditions. Many guidelines and tables have been established but seldom, if ever, have they been supported with empirical evidence.

(4) The peak rate of runoff is sufficient information for the design.

Modern drainage practice often includes detention of urban storm runoff to reduce the peak rate of runoff downstream. With only the peak rate of runoff, the Rational method severely limits the evaluation of design alternatives available in urban and, in some instances, rural drainage design.

7.18.4 Equation

The Rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analyzed). The Rational formula is expressed as follows:

$$Q = CIA \qquad (7.15)$$

where: Q = maximum rate of runoff, ft^3/s

C = runoff coefficient representing a ratio of runoff to rainfall

I = average rainfall intensity for a duration equal to the time of concentration for a selected return period, in/h

A = drainage area tributary to the design location, ha

7.18.5 Infrequent Storm

The coefficients given in Tables 7-9 through 7-11 are applicable for storms of 5-yr to 10-yr frequencies. Less frequent, higher intensity storms will require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff Reference *(19)*. The adjustment of the Rational method for use with major storms can be made by multiplying the right side of the Rational formula by a frequency factor C_f. The Rational formula now becomes:

$$Q = CC_fIA \qquad (7.16)$$

C_f values are listed in Table 7-7.

TABLE 7-7 — Frequency Factors for Rational Formula

Recurrence Interval (years)	C_f
25	1.1
50	1.2
100	1.25

The product of C_f times C shall not exceed 1.0.

TABLE 7-8 — Hydrologic Soils Groups For

Example for Orange County, North Carolina

Series Name	Hydrologic Groups	Series Name	Hydrologic Groups
Altavista	C	Herndon	B
Appling	B	Hiwassee	B
Cecil	B	Iredell	D
Chewacla	C	Lignum	C

TABLE 7-9 — Recommended Coefficient of Runoff for Pervious Surfaces (By Selected Hydrologic Soil Groupings and Slope Ranges)

Slope	A	B	C	D
Flat (0% – 1%)	0.04 – 0.09	0.07 – 0.12	0.11 – 0.16	0.15 – 0.20
Average (2% – 6%)	0.09 – 0.14	0.12 – 0.17	0.16 – 0.21	0.20 – 0.25
Steep (Over 6%)	0.13 – 0.18	0.18 – 0.24	0.23 – 0.31	0.28 – 0.38

Source: (Example from *Storm Drainage Design Manual*, Erie and Niagara Counties Regional Planning Board)

TABLE 7-10 — Recommended Coefficient of Runoff Values
(For Various Selected Land Uses)

Description of Area		Runoff Coefficients
Business: Downtown areas		0.70 – 0.95
Neighborhood areas		0.50 – 0.70
Residential:	Single-family areas	0.30 – 0.50
	Multi units, detached	0.40 – 0.60
	Multi units, attached	0.60 – 0.75
	Suburban	0.25 – 0.40
Residential (0.5 ha lots or more)		0.30 – 0.45
Apartment dwelling areas		0.50 – 0.70
Industrial:	Light areas	0.50 – 0.80
	Heavy areas	0.60 – 0.90
Parks, cemeteries		0.10 – 0.25
Playgrounds		0.20 – 0.40
Railroad yard areas		0.20 – 0.40
Unimproved areas		0.10 – 0.30

Source: Reference *(3)*.

TABLE 7-11 — Coefficients for Composite Runoff Analysis

Surface		Runoff Coefficients
Streets: Asphalt		0.70 – 0.95
	Concrete	0.80 – 0.95
Drives and walks		0.75 – 0.85
Roofs		0.75 – 0.95

Source: Reference *(3)*.

7.18.6 Procedures

The results of using the Rational formula to estimate peak discharges are very sensitive to the parameters used, especially time of concentration and runoff coefficient. The designer must use good engineering judgment in estimating values that are used in the method. Following is a discussion of the different variables used in the Rational method.

7.18.6.1 Time of Concentration

The time of concentration is the time required for water to flow from the hydraulically most remote point of the drainage area to the point under investigation. Use of the Rational formula requires the time of concentration (t_c) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). For a specific drainage basin, the time of concentration consists of an inlet time plus the time of flow in a closed conduit or open channel to the design point. Inlet time is the time required for runoff to flow over the surface to the nearest inlet and is primarily a function of the length of overland flow, the slope of the drainage basin and surface cover. Pipe or open channel flow time can be estimated from the hydraulic properties of the conduit or channel.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length.

Manning's equation can be used to determine velocity. See Chapter 8 for a discussion of Manning's equation.

In some cases, estimates based on an entire watershed can result in lower peak discharges than might result in considering only a portion of the watershed. Such cases may occur with highly irregular watershed shapes, non-homogeneous land use and highly variable slopes.

Development within a watershed may create changes in overland flow paths. Often, the land will be graded and swales will intercept the natural contour and conduct the water to the streets, which reduces the time of concentration. Designers should identify constructed topographic changes and include present flow characteristics in their design.

See Section 7.17 for a discussion on time of concentration.

7.18.6.2 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate (in/h) for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves. An example of such a curve is given in Figure 7-5; other curves for use by the Department are given in Appendix 7.B.

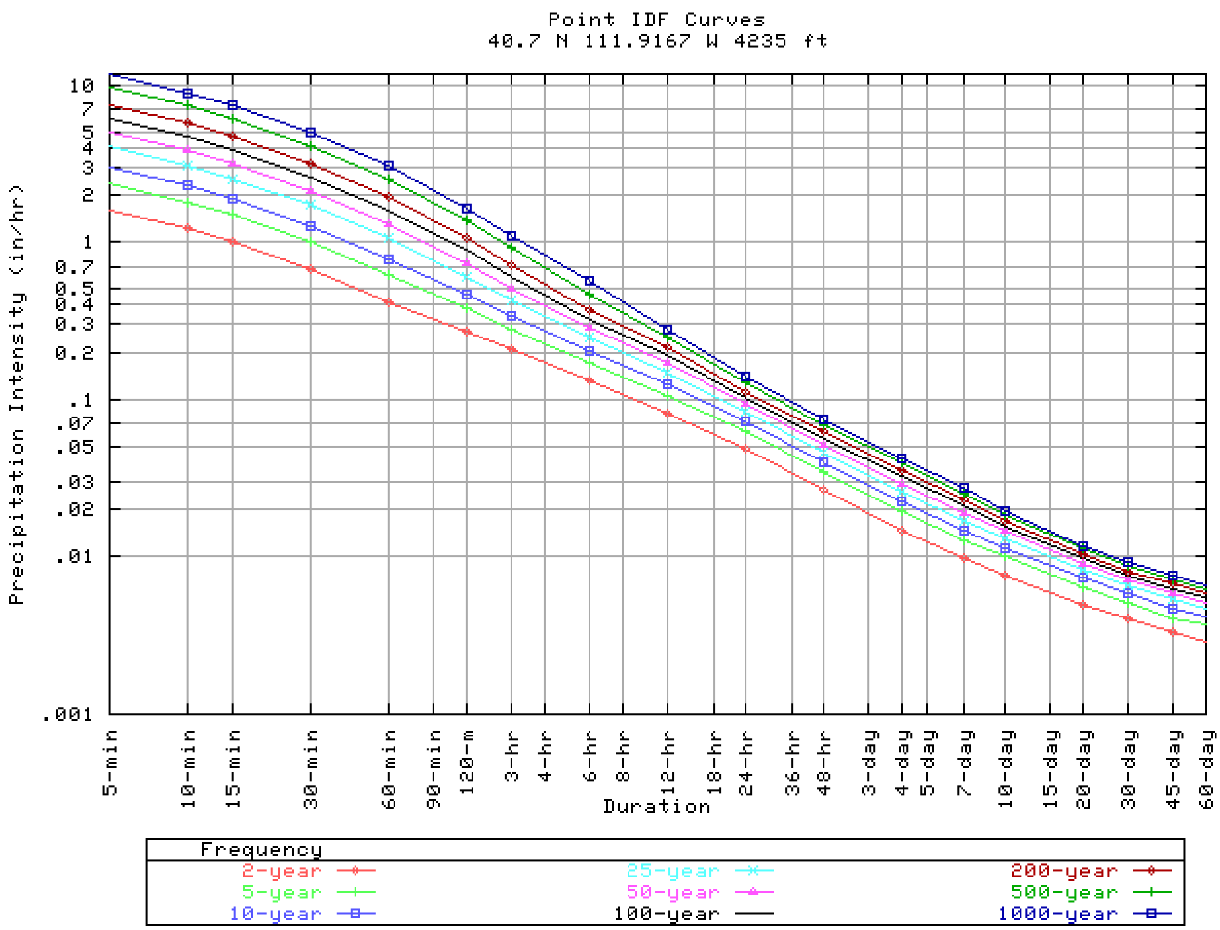


FIGURE 7-5 — Intensity Duration Curve

7.18.6.3 Runoff Coefficient

The runoff coefficient, C, is the variable of the Rational method least amenable to precise determination and requires the judgment and understanding of the designer. Although engineering judgment will always be required in the selection of runoff coefficients, a typical coefficient represents the integrated effects of many drainage basin parameters; the following discussion considers only the effects of soil groups, land use and average land slope.

Three methods for determining the runoff coefficient are presented based on soil groups and land slope (Tables 7-8 and 7-9), land use (Table 7-10) and a composite coefficient for complex watersheds (Table 7-11).

Table 7-9 gives the recommended coefficient, C, of runoff for pervious surfaces by selected hydrologic soil groupings and slope ranges. From this Table, the C values for non-urban areas (e.g., forest land, agricultural land, open space) can be determined. Soil properties influence the relationship between runoff and rainfall because soils have differing rates of infiltration. Infiltration is the movement of water through the soil surface into the soil. Based on infiltration rates, NRCS has divided soils into four hydrologic soil groups as follows:

Group A Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse textures.

Group C Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high watertables, soils with a claypan or clay layer at or near the surface and shallow soils over nearly impervious parent material.

As an example, a list of soils for Orange County, North Carolina and their hydrologic classification is presented in Table 7-8. Other soil classifications important to the Department are given in Appendix 7.C.

As the slope of the drainage basin increases, the selected C value should also increase. This is because, as the slope of the drainage area increases, the velocity of overland and channel flow will increase allowing less opportunity for water to infiltrate the ground surface. Thus, more of the rainfall will become runoff from the drainage area.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. Composites can be made with Tables 7-9 and 7-10. At a more detailed level, composites can be made with Table 7-9 and the coefficients with respect to surface type given in Table 7-11. The composite procedure can be applied to an entire drainage area or to typical “sample” blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

7.19 EXAMPLE PROBLEM - RATIONAL METHOD

Following is an Example problem that illustrates the application of the Rational method to estimate peak discharges.

Preliminary estimates of the maximum rate of runoff are needed at the inlet to a culvert for a 10-yr and 50-yr return period.

Site Data

From a topographic map and field survey, the area of the drainage basin upstream from the point in question is found to be 50 acres. In addition the following data were measured:

Length of overland flow = 260 ft Average overland slope = 0.3%
Length of grassed waterway = 66 ft Average grassed waterway slope = 0.5%

Land Use

From existing land use maps, land use for the drainage basin was estimated to be:

Residential (1.25-acre lots) 80%
Playground 20%

Step 1 Calculate Time of Concentration

For overland flow (short grass pasture) from Table 7-6, K = 0.213:

$V = (0.7)(0.3)^{0.5} = 0.38$ ft/s

$$t_t = \frac{260\ ft}{(0.38\ ft/s)(60\ s/\min)} = 11.4\ \min$$

For grassed waterway (shallow concentrated flow) from Table 7-6, K = 0.457:

$V = (1.5)(0.5)^{0.5} = 1.06$ ft/s

$$t_t = \frac{66\ ft}{(1.06\ ft/s)(60\ s/\min)} = 1.0\ \min$$

$t_c = t_t$ (overland flow) + t_t (grassed waterway) = 12.4 min

Step 2 Determine Rainfall Intensity

From Figure 7-5 with duration equal to 21.5 min:

I_{10} = 6.3 in/h
I_{50} = 7.9 in/h

Step 3 Determine C, Runoff Coefficient

A weighted runoff coefficient C for the total drainage area is determined in the following table by utilizing the values from Table 7-10:

Land Use	(1) Percent of Total Land Area	(2) Weighted Runoff Coefficient	(3) Runoff Coefficient*
Residential (0.5-ha lots)	80%	0.30	0.24
Playground	20%	0.20	0.04
Total Weighted Runoff			0.28

Column 3 equals Column 1 multiplied by Column 2.

Step 4 Determine Peak Runoff

From the Rational equation:

$$Q_{10} = CIA = (0.28)(6.3\ in/h)(2\ acres) = 3.58\ ft^3/s$$

$$Q_{50} = C_f CIA = (1.2)(0.28)(7.9\ in/h)(2.0\ acres) = 5.38\ ft^3/s$$

Note: C_f = 1.2 from Table 7-7.

These are the estimates of peak runoff for a 10-yr and 50-yr design storm for the given basin.

7.20 NRCS UNIT HYDROGRAPH

7.20.1 Introduction

Techniques developed by NRCS for calculating rates of runoff require the same basic data as the Rational method — drainage area, a runoff factor, time of concentration and rainfall. The NRCS approach, however, is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage and an infiltration rate that decreases during the course of a storm. With the NRCS method, the direct runoff can be calculated for any storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. Details of the methodology can be found in Reference *(14)*.

7.20.2 Application

Two types of hydrographs are used in the NRCS procedure, unit hydrographs and dimensionless hydrographs. A unit hydrograph represents the time distribution of flow resulting from 1 in of direct runoff occurring over the watershed in a specified time. A dimensionless hydrograph represents the composite of many unit hydrographs. The dimensionless unit hydrograph is plotted in non-dimensional units of time versus time to peak and discharge at any time versus peak discharge.

Characteristics of the dimensionless hydrograph vary with the size, shape and slope of the tributary drainage area. The most significant characteristics affecting the dimensionless hydrograph shape are the basin lag and the peak discharge for a given rainfall. Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak. Steep slopes, compact shape and an efficient drainage network tend to make lag time short and peaks high; flat slopes, elongated shape and an inefficient drainage network tend to make lag time long and peaks low.

7.20.3 Equations and Concepts

The following discussion outlines the equations and basic concepts utilized in the NRCS method:

DRAINAGE AREA. The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into subdrainage areas to account for major land-use changes, obtain analysis results at different points within the drainage area, or locate stormwater drainage facilities and assess their effects on the flood flows. Also, a field inspection of existing or proposed drainage systems should be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the subdrainage areas.

RAINFALL. The NRCS method is based on a 24-h storm event which has a Type II time distribution. The Type II storm distribution is a “typical” time distribution which the NRCS has prepared from rainfall records for Utah. Table 7-12 illustrates both the Type II and Type III distributions. To use one of these distributions, it is necessary for the user to obtain the 24-h rainfall value (from the figures in Appendix 7.B) for the frequency of the design storm desired.

RAINFALL-RUNOFF EQUATION. A relationship between accumulated rainfall and accumulated runoff was derived by NRCS from experimental plots for numerous soils and vegetative cover conditions. Data for land-treatment measures (e.g., contouring, terracing) from experimental watersheds were included. The Equation was developed mainly for small watersheds for which only daily rainfall and watershed data are ordinarily available. It was developed from recorded storm data that included the total amount of rainfall in a calendar day but not its distribution with respect to time. The NRCS runoff equation is therefore a method of estimating direct runoff from a 24-h or 1-d storm rainfall. The Equation is:

$$Q = (P - I_a)^2 / (P - I_a) + S \qquad (7.17)$$

where:

- Q = accumulated direct runoff, in
- P = accumulated rainfall (potential maximum runoff), in
- I_a = initial abstraction including surface storage, interception and infiltration prior to runoff, in
- S = potential maximum retention, in

The relationship between I_a and S was developed from experimental watershed data. It removes the necessity for estimating I_a for common usage. The empirical relationship used in the NRCS runoff Equation is:

$$I_a = 0.2S \qquad (7.18)$$

Substituting 0.2S for I_a in Equation 7.17, the NRCS rainfall-runoff equation becomes:

$$Q = (P - 0.2S)^2 / (P + 0.8S) \qquad (7.19)$$

Figure 7-6 shows a graphical solution of this Equation, which enables the precipitation excess from a storm to be obtained if the total rainfall and watershed curve number are known.

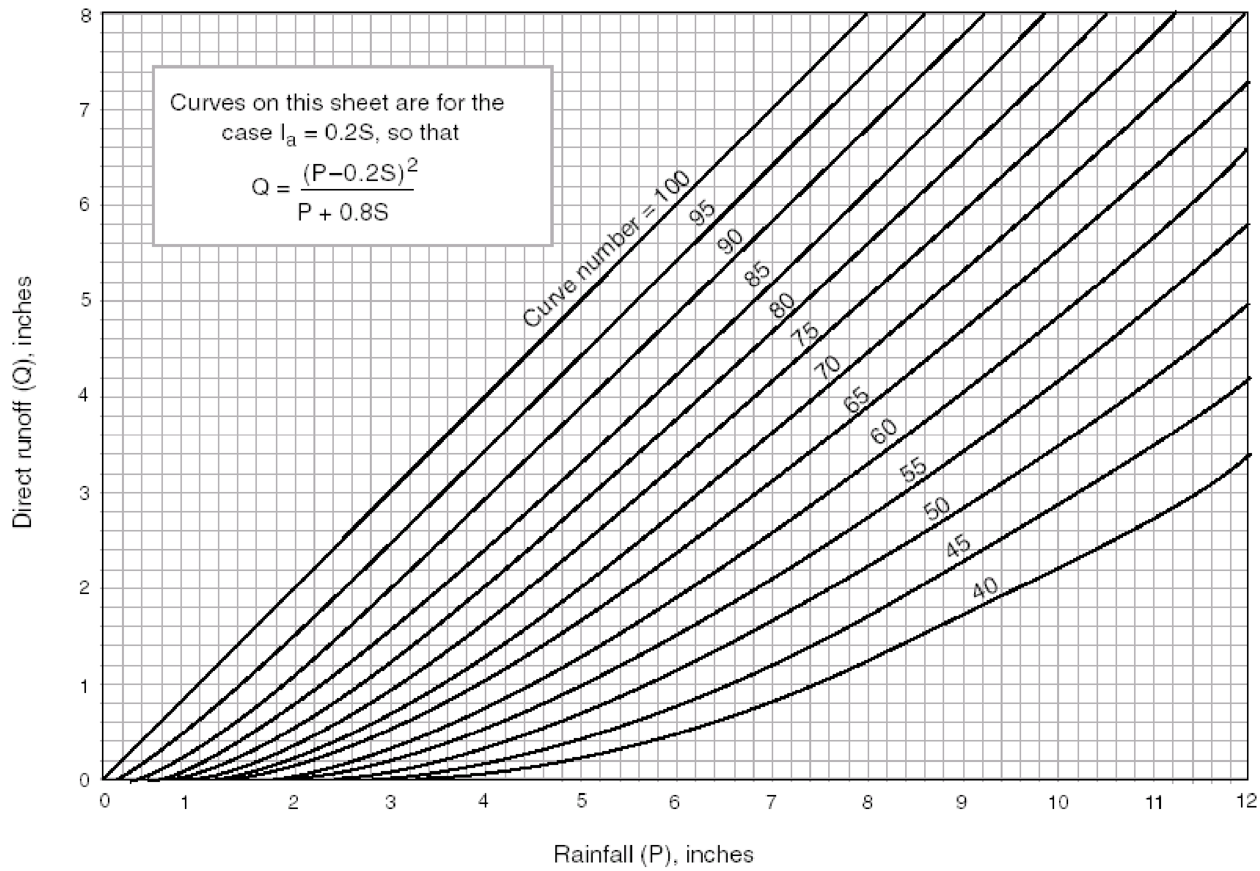


FIGURE 7-6 — NRCS Relation Between Direct Runoff, Curve Number and Precipitation

TABLE 7-12 — NRCS 24-Hour Rainfall Distributions

Note: Table represents the cumulative fraction of rainfall with respect to time.

Time, t (hours)	Fraction of 24-hour Rainfall	
	Type II	Type III
0	0	0
2	0.022	0.020
4	0.048	0.043
6	0.080	0.072
7	0.098	0.089
8	0.120	0.115
8.5	0.133	0.130
9	0.147	0.148
9.5	0.163	0.167
9.75	0.172	0.178
10	0.181	0.189
10.5	0.204	0.216
11	0.235	0.250
11.5	0.283	0.298
11.75	0.393	0.356
12	0.663	0.500
12.5	0.735	0.702
13	0.772	0.751
13.5	0.799	0.785
14	0.820	0.811
16	0.880	0.886
20	0.952	0.957
24	1	1

7.20.4 Procedure

Following is a discussion of procedures that are used in the hydrograph method and recommended tables and figures.

7.20.4.1 Runoff Factor

In hydrograph applications, runoff is often referred to as rainfall excess or effective rainfall, all defined as the amount by which rainfall exceeds the capability of the land to infiltrate or otherwise retain the rainwater. The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types and land slope.

Land use is the watershed cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads and roofs are all part of the land use. Land treatment applies mainly to agricultural land use, and it includes mechanical practices (e.g., contouring, terracing) and management practices (e.g., rotation of crops).

NRCS uses a combination of soil conditions and land use (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area when the soil is not frozen. The higher the CN, the higher the runoff potential.

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. NRCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C and D). These Groups were previously described for the Rational method.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions, defined as the amount of rainfall occurring in a selected period preceding a given storm. In general, the greater the antecedent rainfall, the more direct runoff there is from a given storm. A 5-d period is used as the minimum for estimating antecedent moisture conditions. Antecedent soil moisture conditions also vary during a storm; heavy rain falling on a dry soil can change the soil moisture condition from dry to average to wet during the storm period.

The following pages give a series of tables related to runoff factors. The first tables (Tables 7-13 to 7-16) give curve numbers for various land uses. These tables are based on an average antecedent moisture condition; i.e., soils that are neither very wet nor very dry when the design storm begins. Curve numbers should be selected only after a field inspection of the watershed and a review of zoning and soil maps. Table 7-17 gives conversion factors to convert average curve numbers to wet and dry curve numbers. Table 7-18 gives the antecedent conditions for the three classifications.

7.20.4.2 Time of Concentration

The average slope within the watershed together with the overall length and retardance of overland flow are the major factors affecting the runoff rate through the watershed. In the NRCS method, time of concentration (t_c) is defined to be the time required for water to travel from the most hydraulically distant point in a watershed to its outlet. Lag (L) can be considered as a weighted time of concentration and is related to the physical properties of a watershed (e.g., area, length, slope). NRCS derived the following empirical relationship between lag and time of concentration:

$$L = 0.6\ t_c \qquad (7.20)$$

In small urban areas (less than 2,000 acres or 3.1 square miles), a Curve Number method can be used to estimate the time of concentration from watershed lag. In this method, the lag for the runoff from an increment of excess rainfall can be considered as the time between the center of mass of the excess rainfall increment and the peak of its incremental outflow hydrograph. The equation developed by NRCS to estimate lag is:

$$L = (l^{0.8}\ (S + 1)^{0.7}) / (1900\ Y^{0.5}) \qquad (7.21)$$

where:
- L = lag, h
- l = length of mainstream to farthest divide, ft
- Y = average slope of watershed, %
- S = [(1000/CN) – 10], in
- CN = NRCS Curve Number

TABLE 7-13 — Runoff Curve Numbers - Urban Areas[1]

Cover Description	Curve Numbers for Hydrologic Soil Groups				
Cover Type and Hydrologic Condition	Average Percent Impervious Area[2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-in sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
0.12 acres or less (townhouses)	65	77	85	90	92
0.25 acres	38	61	75	83	87
0.33 acres	30	57	72	81	86
0.5 acres	25	54	70	80	85
1.0 acres	20	51	68	79	84
2.0 acres	12	46	65	77	82
Developing urban areas:					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94
Idle lands (CNs are determined using cover types similar to those in Table 7-16).					

[1] Average runoff condition and $I_a = 0.2S$.

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the NRCS method has an adjustment to reduce the effect.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

TABLE 7-14 — Cultivated Agricultural Land[1]

Cover Description			Curve Numbers for Hydrologic Soil Group			
Cover Type	Treatment[2]	Hydrologic Condition[3]	A	B	C	D
Fallow	Bare soil	—	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row Crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & Terraced (C & T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
	Small grain SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
	Close-seeded SR or broadcast	Poor	66	77	85	89
		Good	58	72	81	85
	Legumes or C Rotation	Poor	64	75	83	85
		Good	55	69	78	83
	Meadow C&T	Poor	63	73	80	83
		Good	51	67	76	80

1 Average runoff condition and $I_a = 0.2S$.

2 Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

3 Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or closed-seeded legumes in rotations, (d) percent of residue cover on the land surface (good > 20%), and (e) degree of roughness.

Poor: Factors impair infiltration and tend to increase runoff.
Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

TABLE 7-15 — Other Agricultural Lands[1]

Cover Description	Hydrologic Condition	Curve Numbers for Hydrologic Soil Group			
Cover Type		A	B	C	D
Pasture, grassland, or range — continuous forage for graving [2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow — continuous grass — protected from grazing and generally mowed for hay		30	58	71	78
Brush — brush-weed-grass mixture with brush the major element [3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[4]	48	65	73
Woods — grass combination (orchard or tree farm) [5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[4]	55	70	77
Farmsteads — buildings, land, driveways and surrounding lots	—	59	74	82	86

1 Average runoff condition and $I_a = 0.2S$.

2 Poor: < 50% ground cover or heavily grazed with no mulch
Fair: 50% to 75% ground cover and not heavily grazed
Good: > 75% ground cover and lightly or only occasionally grazed

3 Poor: < 50% ground cover
Fair: 50% to 75% ground cover
Good: > 75% ground cover

4 Actual Curve Number is less than 30; use CN = 30 for runoff computations.

5 CNs shown were computed for areas with 50% grass (pasture) cover. Other combinations of conditions may be computed from CNs for woods and pasture.

6 Poor: Forest litter, small trees and brush are destroyed by heavy grazing or regular burning.
Fair: Woods grazed but not burned, and some forest litter covers the soil.
Good: Woods protected from grazing; litter and brush adequately cover soil.

TABLE 7-16 — Arid and Semiarid Rangelands[1]

Cover Type	Hydrologic Condition[2]	A[3]	B	C	D
Herbaceous — mixture of grass, weeds and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen — mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper — pinyon, juniper or both; grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub — major plants include saltbush, grasswood, creosote-bush, blackbrush, bursage, palo verde, mesquite and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

[1] Average runoff condition and $I_a = 0.2S$

[2] Poor: < 30% ground cover (litter, grass and brush overstory)
Fair: 30% to 70% ground cover
Good: > 70% ground cover

[3] Curve Numbers for Group A have been developed only for desert shrub

TABLE 7-17 — Conversion From Average Antecedent Moisture Conditions To Dry and Wet Conditions

CN For Average Conditions	Corresponding CNs For	
	Dry	Wet
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13

Source: Reference *(13).*

TABLE 7-18 — Rainfall Groups For Antecedent Soil Moisture Conditions During Growing And Dormant Seasons

Antecedent Condition	Condition's Description	Growing Season 5-d Antecedent Rainfall	Dormant Season 5-d Antecedent Rainfall
Dry	An optimum condition of watershed soils, where soils are dry but not to the wilting point and when satisfactory plowing or cultivation takes place	Less than 1.5 in	Less than 0.5 in
Average	The average case for annual floods	1.5 in to 2 in	0.5 in to 1 in
Wet	When a heavy rainfall, or light rainfall and low temperatures, have occurred during the five days previous to a given storm	Over 2 in	Over 1 in

Source: Reference *(14).*

The lag time can be corrected for the effects of urbanization by using Figures 7-7 and 7-8. The amount of modifications to the hydraulic flow length usually must be determined from topographic maps or aerial photographs following a field inspection of the area. The modification to the hydraulic flow length not only includes pipes and channels but also the length of flow in streets and driveways.

After the lag time is adjusted for the effects of urbanization, Equation 7.21, which relates lag time and time of concentration, can be used to calculate the time of concentration for use in the NRCS method. Section 7.17 discusses alternative procedures for travel time and time of concentration estimation.

7.20.4.3 Triangular Hydrograph Equation

The triangular hydrograph is a practical representation of excess runoff with only one rise, one peak and one recession. Its geometric makeup can be easily described mathematically, which makes it very useful in the processes of estimating discharge rates. NRCS developed the following Equation to estimate the peak rate of discharge for an increment of runoff:

$$q_p = 484(q) / (d/2 + L) \tag{7.22}$$

where:
- q_p = peak rate of discharge for unit hydrograph, ft^3/s
- A = area, mi^2
- q = storm runoff during time interval = 1 in for unit hydrograph, in
- d = time interval, h
- L = watershed lag, h

This Equation can be used to estimate the peak discharge for the unit hydrograph, which can then be used to estimate the peak discharge and hydrograph from the entire watershed.

The constant 484, or peak rate factor, is valid for the NRCS dimensionless unit hydrograph. Any change in the dimensionless unit hydrograph reflecting a change in the percent of volume under the rising side would cause a corresponding change in the shape factor associated with the triangular hydrograph and, therefore, a change in the constant 484. This constant has been known to vary from approximately 600 in steep terrain to 300 in very flat swampy country.

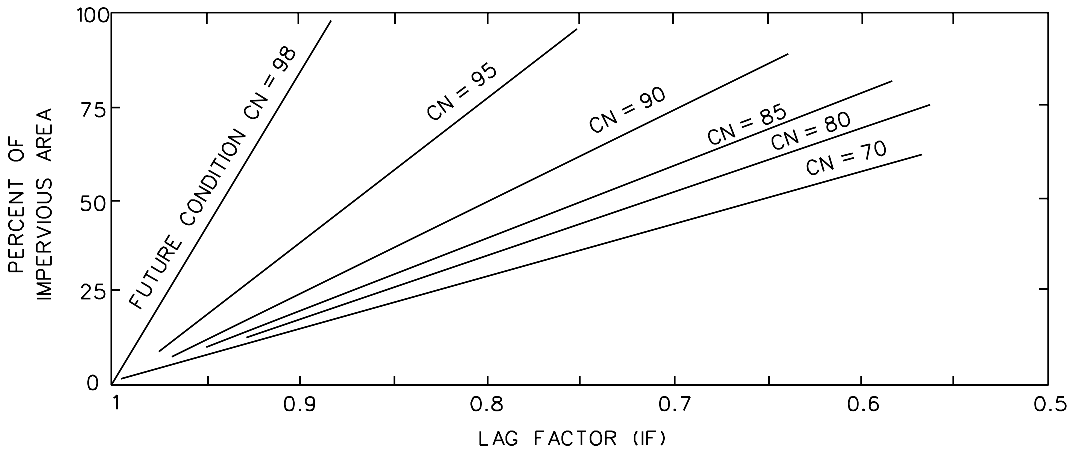


FIGURE 7-7 — Factors For Adjusting Lag When Impervious Areas Occur In Watershed

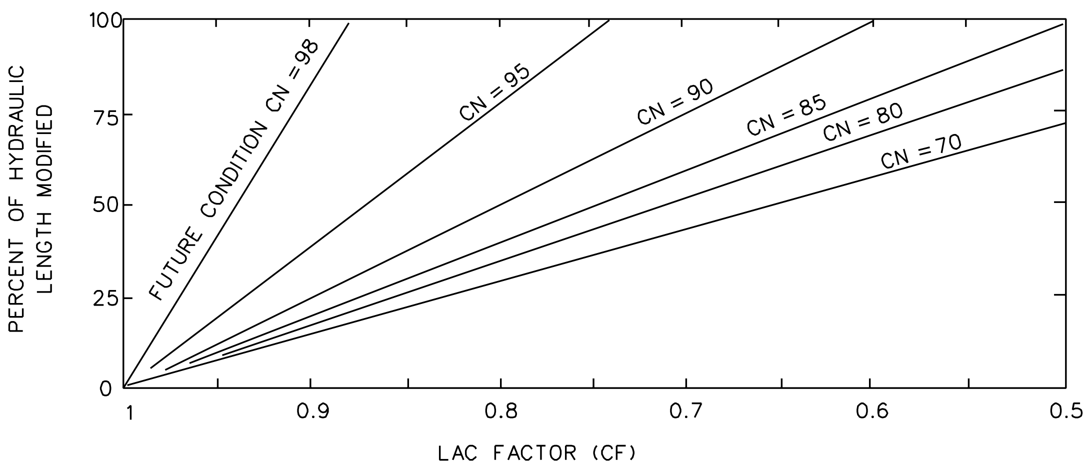


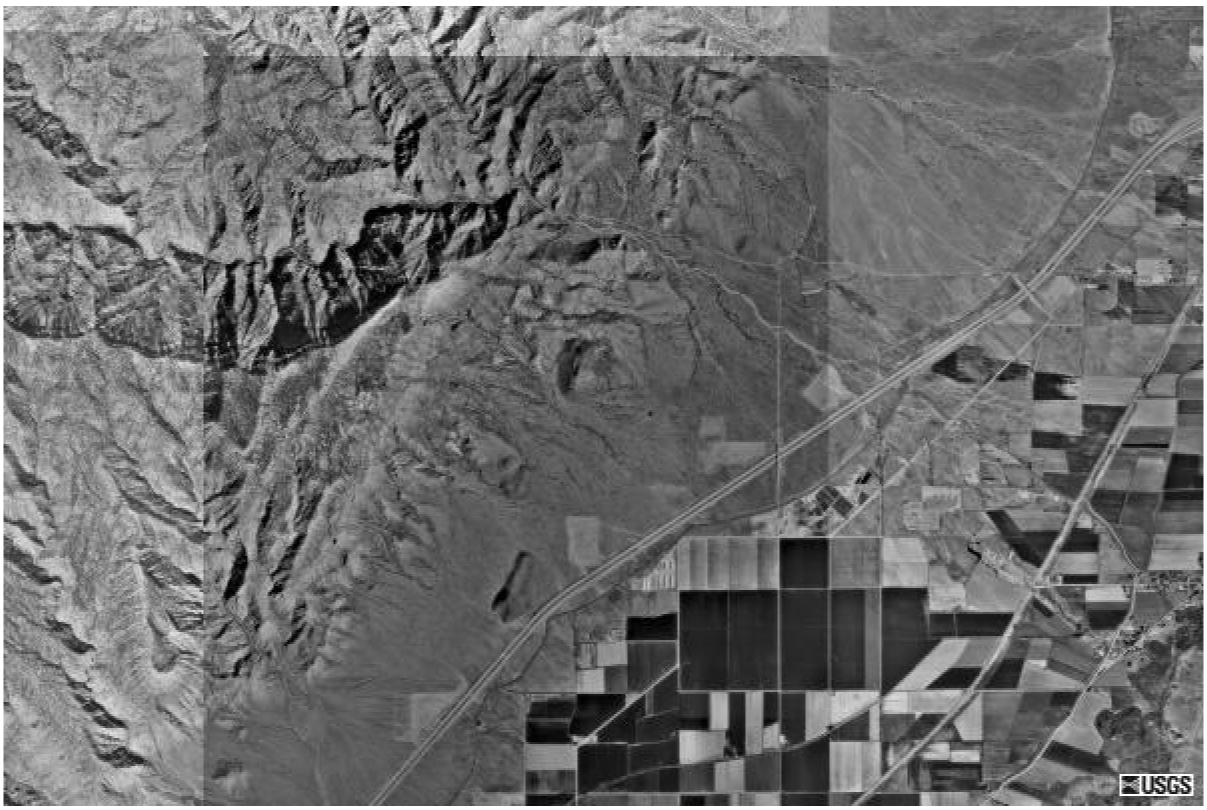
FIGURE 7-8 — Factors For Adjusting Lag When The Main Channel Has Been Hydraulically Improved

Source: Reference *(3).*

7.21 EXAMPLE PROBLEM(S) — NRCS UNIT HYDROGRAPH

Following is an Example problem that illustrates the application of the NRCS Unit Hydrograph to estimate peak discharges.

The concentration point is on the South Cedar Canyon Creek at the north-east edge of the borrow pit east of I-70, north of Richfield, Utah.



Import the Digital Elevation Model (DEM) for the drainage basin area into WMS (Watershed Modeling System). Import the image showing the basin area. While in the DEM module, pull down the drainage menu, compute the Topaz to delineate the drainage paths. Create an outlet point on the drainage path. From the drainage menu, define the basin, change the basins to polygons and compute the drainage data.

The drainage area should be 25.48 square miles.

Set WMS to the Tree module. Use the Select Basin tool and select the basin. Pull down the HEC1 menu and complete the job control. From the HEC1 menu, pick: Edit the HEC1 parameters. Press:

Basin Data – Verify the correct area

Precipitation – Enter the average elevation for the 100 year 24 hour storm (from NOA atlases), 3.4 inches. Press: Define Series, import the NRCS hydrograph series from WMS HEC1 tutorial, and pick type II 24 hour rainfall series.

Press: Loss Method, enter the NRCS curve number for the area. The engineer should sample the soil and test its properties. The soil in this area is classified as AASHTO A-4(8)Max. It is a type C because of the high content of clay. The land is arid and covered with Junipers, Pinyon Pines and Sage Brushes in the undercover. The cover is Fair. Use 68 as a curve number. The ground is 2% to 5% impervious, use 3%.

Press Unit Hydrograph and Computer Parameters-Basin Data. Toggle SCS method and press O.K. twice. Press: Done after entering all the data.

Pull down the HEC1 menu and run HEC1. Give a name to the file and save it in a convenient directory.

Following is a portion of the HEC1 output file:

```
PEAK FLOW     TIME

                              MAXIMUM AVERAGE FLOW

                              6-HR     24-HR     72-HR    37.25-HR

+  (CFS)    (HR) +   2943.    14.25

                 (CFS)        1843.     661.      429.      429.

              (INCHES)        0.672     0.965     0.972     0.972

               (AC-FT)        914.      1311.     1320.     1320.

               CUMULATIVE AREA =  25.48 SQ MI

                                  RUNOFF SUMMARY

                             FLOW IN CUBIC FEET PER SECOND

                           TIME IN HOURS,  AREA IN SQUARE MILES

                  PEAK   TIME OF    AVERAGE FLOW FOR MAXIMUM PERIOD  BASIN

 MAXIMUM TIME OF   OPERATION     STATION    FLOW    PEAK    AREA     STAGE     MAX STAGE

+                                 6-HOUR    24-HOUR    72-HOUR

     HYDROGRAPH AT

+                  1B    2943.  14.25     1843.     661.     429.    25.48

     ROUTED TO

+                  2R    2943.  14.25     1843.     661.     429.    25.48

 *** NORMAL END OF HEC-1 ***
```

7.22 NRCS GRAPHICAL PEAK DISCHARGE METHOD

For many peak discharge estimation methods, the input includes variables to reflect the size of the contributing area, the amount of rainfall, the potential watershed storage and the time-area distribution of the watershed. These are often translated into input variables such as the drainage area, the depth of rainfall, an index reflecting land use and soil type, and the time of concentration. The NRCS Graphical peak discharge method is typical of many peak discharge methods that are based on input such as that described.

7.22.1 Runoff Depth Estimation

The volume of storm runoff can depend on a number of factors. Certainly, the volume of rainfall will be an important factor. For very large watersheds, the volume of runoff from one storm event may depend on rainfall that occurred during previous storm events. However, when using the design storm approach, the assumption of storm independence is quite common.

In addition to rainfall, other factors affect the volume of runoff. A common assumption in hydrologic modeling is that the rainfall available for runoff is separated into three parts — direct (or storm) runoff, initial abstraction and losses. Factors that affect the split between losses and direct runoff include the volume of rainfall, land cover and use, soil type and antecedent moisture conditions. Land cover and land use will determine the amount of depression and interception storage.

7.22.2 Cover Complex Classification

The NRCS cover complex classification consists of three factors — land use, treatment or practice, and hydrologic condition. Many different land uses are identified in the tables for estimating runoff curve numbers. Agricultural land uses are often subdivided by treatment or practices, such as contoured or straight row; this separation reflects the different hydrologic runoff potential that is associated with variation in land treatment. The hydrologic condition reflects the level of land management; it is separated into three classes — poor, fair and good. Not all of the land uses are separated by treatment or condition.

7.22.3 Curve Number Tables

Table 7-15 shows the NRCS CN values for the different land uses, treatments and hydrologic conditions; separate values are given for each soil group. For example, the CN for a wooded area with good cover and soil group B is 55; for soil group C, the CN would increase to 70. If the cover (on soil group B) is poor, the CN will be 66.

7.22.4 Estimation of CN Values for Urban Land Uses

The CN table (Table 7-13) includes CN values for a number of urban land uses. For each of these, the CN is based on a specific percentage of imperviousness. For example, the CN values for commercial land use are based on an imperviousness of 85%. Curve numbers for other percentages of imperviousness can be computed using a weighted CN approach, with a CN of 98 used for the impervious areas and the CN for open space (good condition) used for the pervious portion of the area. Thus, CN values of 39, 61, 74 and 80 are used for hydrologic soil groups A, B, C and D, respectively. These are the same CN values for pasture in good condition. Thus, the following Equation can be used to compute a weighted CN:

$$CN_w = CN_p(1-f) + f(98) \tag{7.23}$$

in which f is the fraction (not percentage) of imperviousness. To show the use of Equation 7.23, the CN values for commercial land use with 85% imperviousness are:

A soil: 39(0.15) + 98(0.85) = 89
B soil: 61(0.15) + 98(0.85) = 92
C soil: 74(0.15) + 98(0.85) = 94
D soil: 80(0.15) + 98(0.85) = 95

These are the same values shown in Table 7-13.

Equation 7.23 can be placed in graphical form (see Figure 7-9a). By entering with the percentage of imperviousness on the vertical axis at the center of the Figure and moving horizontally to the pervious area CN, the composite CN can be read. The examples above for commercial land use can be used to illustrate the use of Figure 7-9a for a 85% imperviousness. For a commercial land area with 60% imperviousness of a B soil, the composite CN would be:

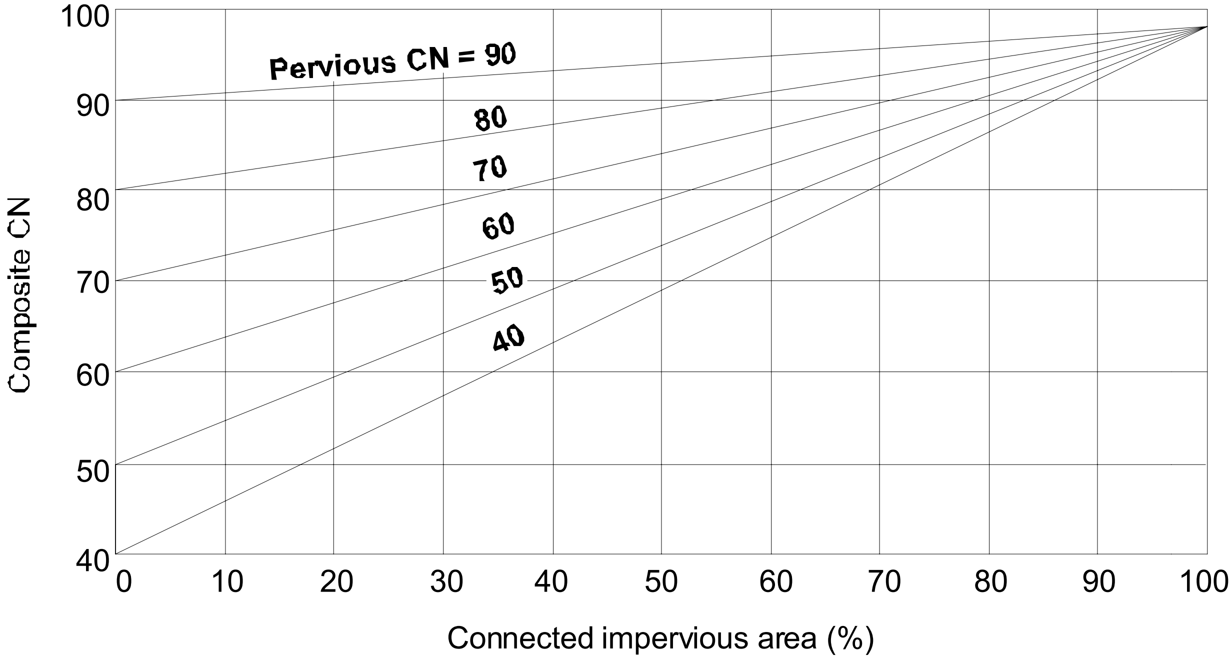
$$CN_w = 61(0.4) + 98(0.6) = 83$$

The same value can be obtained from Figure 7-9a.

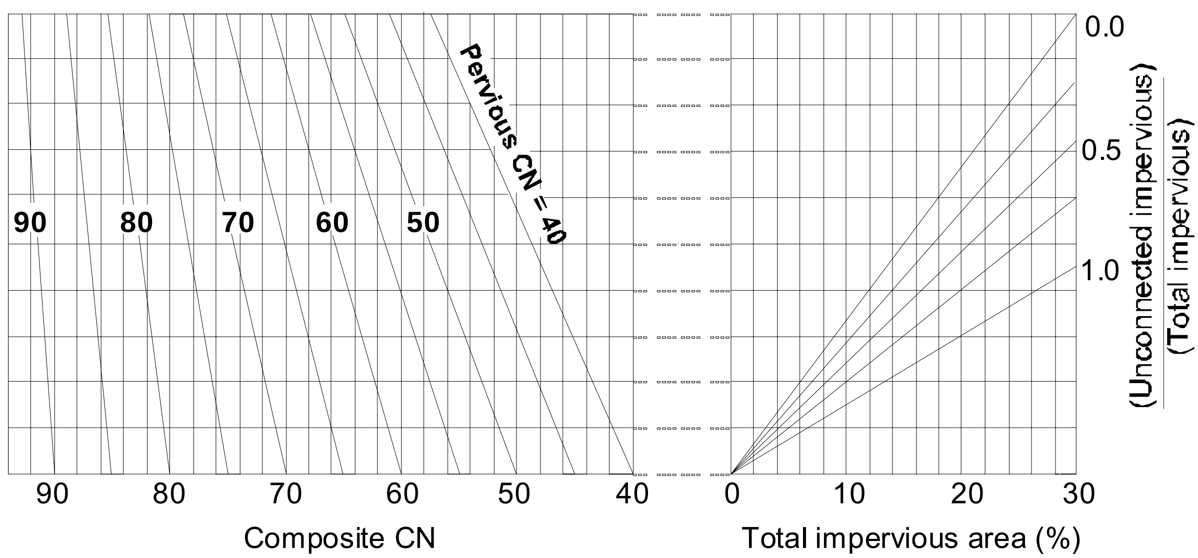
7.22.5 Effect of Unconnected Impervious Area on Curve Numbers

Many local drainage policies are requiring runoff that occurs from certain types of impervious land cover (i.e., rooftops, driveways, patios) to be directed to pervious surfaces rather than being connected to storm drain systems. Such a policy is based on the belief that disconnecting these impervious areas will require smaller and less costly drainage systems and lead both to increased groundwater recharge and to improvements in water quality. If disconnecting some impervious surfaces will reduce both the peak runoff rates and volumes of direct flood runoff, credit should be given in the design of drainage systems. The effect of disconnecting impervious surfaces on runoff rates and volumes can be accounted for by modifying the CN.

There are three variables involved in the adjustment — the pervious area CN, the percentage of impervious area and the percentage of the imperviousness that is unconnected. Because Figure 7-9a for computing composite CN values is based on the pervious area CN and the percentage of imperviousness, a correction factor was developed to compute the composite CN. The correction is a function of the percentage of unconnected imperviousness, which is shown in Figure 7-9b. The use of the correction is limited to drainage areas having percentages of imperviousness that are less than 30%.



(a)



C7

(b)

FIGURE 7-9 — Composite Curve Number Estimation
(a) all imperviousness area connected to storm drains
(b) some imperviousness area not connected to storm drain.

As an alternative to Figure 7-9b, the composite curve number (CN_c) can be computed by:

$$CN_c = CN_p + (P_i / 100)(98 - CN_p)(1 - 0.5R) \quad \text{for } P_i \leq 30\% \tag{7.24}$$

in which P_i is the percent imperviousness and R is the ratio of unconnected impervious area to the total impervious area. Equation 7.24, like Figure 7-9b, is limited to where the total imperviousness (P_i) is less than 30%.

7.22.6 I_a/P Parameter

I_a/P is a parameter that is necessary to estimate peak discharge rates. I_a denotes the initial abstraction, and P is the 24-h rainfall depth for a selected return period. The I_a/P value can be obtained from Table 7-19 for a given CN and P. For a given 24-h rainfall distribution, I_a/P represents the fraction of rainfall that must occur before runoff begins.

7.22.7 Peak Discharge Estimation

The following Equation can be used to compute a peak discharge with the NRCS method:

$$q_p = q_u A Q \tag{7.25}$$

in which q_p is the peak discharge in ft^3/s, q_u the unit peak discharge in $ft^3/s/mi^2/in$ of runoff, A is the drainage area in square kilometers, and Q is the depth of runoff in mm.

The unit peak discharge is obtained from the following Equation, which requires the time of concentration (T_c) in hours and the initial abstraction/rainfall (I_a/P) ratio as input:

$$q_u = \left(10^{c_o + c_1 \log T_c + c_2 [\log(T_c)]^2}\right) \tag{7.26}$$

in which the values of C_o, C_1 and C_2 are given in Table 7-20 for various I_a/P ratios. The runoff depth (Q) is obtained from Equation 7.17 and is a function of the depth of rainfall P and the runoff CN. The I_a/P ratio is obtained either directly by $I_a = 0.2S$ or from Table 7-19; the ratio is a function of the CN and the depth of rainfall.

The peak discharge obtained from Equation 7.25 assumes that the topography is such that surface flow into ditches, drains and streams is relatively unimpeded. Where ponding or swampy areas occur in the watershed, a considerable amount of the surface runoff may be retained in temporary storage. The peak discharge rate should be reduced to reflect this condition of increased storage. Values of the pond and swamp adjustment factor (F_p) are provided in Table 7-21. The adjustment factor values in Table 7-21 are a function of the percent of the total watershed area in ponds and swamps (PPS). If the watershed includes significant portions of pond and swamp storage, then the peak discharge of Equation 7.25 can be adjusted using the following:

$$q_a = q_p F_p \tag{7.27}$$

in which q_a is the adjusted peak discharge in ft^3/s.

TABLE 7-19 — I_a/P for Selected Rainfall Depths and Curve Numbers

Rainfall (in)	Curve Number											
	40	45	50	55	60	65	70	75	80	85	90	95
0.4	*	*	*	*	*	*	*	*	*	*	*	.27
0.8	*	*	*	*	*	*	*	*	*	.45	.28	.13
1.2	*	*	*	*	*	*	*	*	.42	.30	.19	+
1.6	*	*	*	*	*	*	*	.42	.32	.22	.14	+
2.0	*	*	*	*	*	*	.44	.34	.25	.18	.11	+
2.4	*	*	*	*	*	.46	.36	.28	.21	.15	+	+
2.8	*	*	*	*	.48	.39	.31	.24	.18	.13	+	+
3.2	*	*	*	*	.42	.34	.27	.21	.16	.11	+	+
3.6	*	*	*	.46	.38	.30	.24	.19	.14	.10	+	+
4.0	*	*	*	.42	.34	.27	.22	.17	.13	+	+	+
4.4	*	*	.46	.38	.31	.25	.20	.15	.12	+	+	+
4.8	*	*	.42	.35	.28	.23	.18	.14	.11	+	+	+
5.2	*	.48	.39	.32	.26	.21	.17	.13	.10	+	+	+
5.6	*	.44	.36	.30	.24	.20	.16	.12	+	+	+	+
6.0	*	.41	.34	.28	.23	.18	.15	.11	+	+	+	+
6.4	.48	.39	.32	.26	.21	.17	.14	.11	+	+	+	+
6.8	.45	.37	.30	.24	.20	.16	.13	.10	+	+	+	+
7.2	.42	.34	.28	.23	.19	.15	.12	+	+	+	+	+
7.6	.40	.33	.27	.22	.18	.14	.11	+	+	+	+	+
8.0	.38	.31	.25	.21	.17	.14	.11	+	+	+	+	+
8.4	.36	.30	.24	.20	.16	.13	.10	+	+	+	+	+
8.8	.35	.28	.23	.19	.15	.12	.10	+	+	+	+	+
9.2	.33	.27	.22	.18	.15	.12	+	+	+	+	+	+
9.6	.32	.26	.21	.17	.14	.11	+	+	+	+	+	+
10.0	.30	.25	.20	.17	.14	.11	+	+	+	+	+	+
10.4	.29	.24	.20	.16	.13	.11	+	+	+	+	+	+
10.8	.28	.23	.19	.15	.13	.10	+	+	+	+	+	+
11.0	.27	.22	.18	.15	.12	.10	+	+	+	+	+	+
11.4	.26	.21	.18	.14	.12	+	+	+	+	+	+	+
11.8	.25	.21	.17	.14	.11	+	+	+	+	+	+	+
12.2	.25	.20	.16	.13	.11	+	+	+	+	+	+	+
12.6	.24	.19	.16	.13	.11	+	+	+	+	+	+	+
13.0	.23	.19	.15	.13	.10	+	+	+	+	+	+	+
13.4	.22	.18	.15	.12	.10	+	+	+	+	+	+	+
13.8	.22	.18	.15	.12	.10	+	+	+	+	+	+	+
14.2	.21	.17	.14	.12	+	+	+	+	+	+	+	+
14.6	.21	.17	.14	.11	+	+	+	+	+	+	+	+
15.0	.20	.16	.13	.11	+	+	+	+	+	+	+	+
15.4	.20	.16	.13	.11	+	+	+	+	+	+	+	+
15.7	.19	.16	.13	.10	+	+	+	+	+	+	+	+

* signifies that $I_a/P = 0.50$ should be used
+ signifies that $I_a/P = 0.10$ should be used

TABLE 7-20 — Coefficients for NRCS Peak Discharge Method (Equation 7.26)

Rainfall type	I_a/P	C_0	C_1	C_2
I	0.10	2.30550	-0.51429	-0.11750
	0.20	2.23537	-0.50387	-0.08929
	0.25	2.18219	-0.48488	-0.06589
	0.30	2.10624	-0.45695	-0.02835
	0.35	2.00303	-0.40769	0.01983
	0.40	1.87733	-0.32274	0.05754
	0.45	1.76312	-0.15644	0.00453
	0.50	1.67889	-0.06930	0.0
IA	0.10	2.03250	-0.31583	-0.13748
	0.20	1.91978	-0.28215	-0.07020
	0.25	1.83842	-0.25543	-0.02597
	0.30	1.72657	-0.19826	0.02633
	0.50	1.63417	-0.09100	0.0
II	0.10	2.55323	-0.61512	-0.16403
	0.30	2.46532	-0.62257	-0.11657
	0.35	2.41896	-0.61594	-0.08820
	0.40	2.36409	-0.59857	-0.05621
	0.45	2.29238	-0.57005	-0.02281
	0.50	2.20282	-0.51599	-0.01259
III	0.10	2.47317	-0.51848	-0.17083
	0.30	2.39628	-0.51202	-0.13245
	0.35	2.35477	-0.49735	-0.11985
	0.40	2.30726	-0.46541	-0.11094
	0.45	2.24876	-0.41314	-0.11508
	0.50	2.17772	-0.36803	-0.09525

TABLE 7-21 — Adjustment Factor (F_p) for Pond and Swamp Areas that are Spread Throughout the Watershed

Area of pond and swamp (%)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

The NRCS method has a number of limitations. When these conditions are not met, the accuracy of estimated peak discharges decreases. The method should be used on watersheds that are homogeneous in CN; where parts of the watershed have CNs that differ by 5, the watershed should be subdivided and analyzed using a hydrograph method such as TR-20. The NRCS methods should be used only when the CN is 50 or greater and the T_c is greater than 0.1 h and less than 10 h. Also, the computed value of I_a/P should be between 0.1 and 0.5. The method should be used only when the watershed has one main channel or where there are two main channels that have nearly equal times of concentration; otherwise, a hydrograph method should be used. Other methods should also be used where channel or reservoir routing is required or where watershed storage is either greater than 5% or located on the flow path used to compute the T_c.

7.23 EXAMPLE PROBLEM — NRCS GRAPHICAL PEAK DISCHARGE METHOD

Following is an Example problem that illustrates the application of the NRCS Graphical Peak Discharge Method to calculated peak discharges.

A small watershed (17.6 acre) is being developed and will include the following land uses: 9.6 ha of residential (0.25-acre lots) and 8.0 acre of residential (0.5-acre lots). The development will necessitate upgrading of the drainage of a local roadway at the outlet of the watershed. The peak discharge for a 10-yr return period is determined using the NRCS Graphical Method.

The weighted CN is computed using the CN values of Table 7-13:

Land Cover	Area A (acre)	Soil Group	CN	(A)(CN)
Residential (0.5-acre lots)	8.0	C	80	640
Residential (0.25-acre lots)	9.6	B	75	720
	17.6			1360

The weighted CN is:

$$CN_w = \frac{\sum (A)(CN)}{\sum A} = \frac{1360}{17.6} = 77.3 \quad \text{(use 77)}$$

The time of concentration is computed using the velocity method for conditions along the principal flowpath:

Land Cover	Slope (%)	Length (ft)	k for $V = kS^{0.5}$	V (ft/s)	T_t (h)
Woodland (overland	2.3	82	0.5	0.076	0.30
Grassed waterway	2.1	275	1.5	0.217	0.35
Grassed waterway	1.8	250	1.5	0.20	0.35
Concrete-lined channel	1.8	50	—	15.4	0.0009
		657			1.00

The velocity was computed for the concrete-lined channel using Manning's equation, with n = 0.013 and hydraulic radius of 1 ft. The sum of the travel times for the principal flowpath is 0.261 hours.

The rainfall depth is obtained from an IDF curve for the locality using a duration of 24 h and a 10-yr return period. (Note that the T_c is not used to find the rainfall depth when using the NRCS Graphical method. A duration of 24 h is used.) For this Example, a 10-yr rainfall depth of 4.8 in is assumed. For a CN of 77, S = 3.0 in and I_a = 0.6 in. Thus, I_a/P = 0.13. The rainfall depth is computed with Equation 7.19:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} = \frac{(4.8 - 0.2(3))^2}{4.8 + 0.8(3)} = 2.45in$$

The unit peak discharge is computed with Equation 7.26:

$$\begin{aligned} q_u &= \left(10^{2.553 - 0.6151\log(1) - 0.164[\log(1)]^2}\right) \\ &= 357\ ft^3/s/mi^2/in \end{aligned}$$

Thus, the peak discharge is:

$$\begin{aligned} q_p &= q_u AQ = 357\ ft^3/s/mi^2/in(0.068\ mi^2)(2.45\ in) \\ &= 59.5\ ft^3/s \end{aligned}$$

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